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MAY 1960 VOL. LV. NO. 5

FIFTY-FIFTH YEAR OF PUBLICATION

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LEADING CONTENTS

	PAGE
The Earnings of Engineers	185
Some Recent Works of Professor P. L. Nervi	188
Stresses and Factors of Safety in Prestressed Concrete *By P. W. Abeles, D.Sc., M.I.Struct.E.*	191
New Large Bridges in Australia	195
Slabs Spanning in Two Directions Designed by the Yield-line Method *By G. M. Mills, B.Sc.Tech., A.M.I.C.E.*	205
The Approach to Blackwall Tunnel	209
Code of Practice for Road Bridges in India	213
Concreting in Hot Weather	215

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INDEX OF ADVERTISERS

	Page
A.B. Mould & Construction Co. Ltd.	liv
Aberdeen & Co. (Engineers) Ltd.	xxx
A.C.E. Machinery Ltd.	xxxix
Acrow (Engineers) Ltd.	lxxvii
Allan, R. P., & Co. Ltd.	lxxiii
Anderson & Co. Ltd.	xxxviii
Berry's Engineering Co.	viii
B. & M. Reinforcements	lxxxvii
Boyer, Wm., & Sons Ltd.	lvi
Braithwaite Foundations & Construction Ltd.	ii
B.R.C. Engineering Co. Ltd.	xv
British Steel Piling Co. Ltd.	lix
Byrd, A. A., & Co. Ltd.	lxxv
Caplin Engineering Co. Ltd.	lxiii
Cement Marketing Co. Ltd.	lxxvii
Cement & Steel Ltd.	iv
Chapman, H. W., Ltd.	—
Cherriman, Alex. J., Ltd.	xxxvii
Chemical Building Products Ltd.	lxxi
Christiani & Nielsen Ltd.	—
Cochrane, John, & Sons Ltd.	—
Colcrete Ltd.	xxiii
Colvilles Ltd.	lxxvi
Compactors Engineering Ltd.	xii
Concrete Piling Ltd.	xv
Concrete Proofing Co. Ltd.	xxx
Concrete Pump Co. Ltd.	xxxvi
Concrete Services Ltd.	—
Conveyancer & Co. Ltd.	lxx
Cornelly Equipment Co. Ltd.	—
D.A. Moulds Ltd.	vi
Davidson Paper Sales Ltd.	—
Denison, Sam., & Son Ltd.	—
Dickens, Stuart B., Ltd.	xxxv
Dorman Long & Co. Ltd.	—
Duncan Logan (Contractors) Ltd.	—
Engineering Design & Construction Ltd.	xvii
Evode Ltd.	xxiii, xiii, xiii, lvi
Expanded Metal Co. Ltd.	xvii
Expandite Ltd.	iii
Fed (Great Britain) Ltd.	ii
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Kennedy-Greenwood & Co. Ltd.	xxv
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Kinnear Moodie & Co. Ltd.	—
Lafarge Aluminous Cement Co. Ltd.	xxx, lxxxii
Lawe & Son (Stanford) Ltd.	lxiv
Lewis Oil & Grease Co. Ltd.	xliv
Limecrete Ltd.	xliv
Lillington, George, & Co. Ltd.	xlvi
Lind, Peter, & Co. Ltd.	vii
McCall & Co. (Sheffield) Ltd.	ii
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Prestressed Sales Ltd.	—
Priest, B., & Sons Ltd.	xlviii
Pumpcrete Hire Co. Ltd.	xliii
Purpoint Ltd.	lxii
Ransalah Steel Moulds Ltd.	liv
Rawlplug Co. Ltd.	iii
Reed & Mallik Ltd.	—
Rover Car Co. Ltd.	lxviii
Rush & Tompkins Ltd.	—
Rylands Brothers Ltd.	lxvi
Samuel & Son Ltd.	lxxii
Sealocrete Products Ltd.	xiv
Semco-Carves Ltd.	xiii
Simplex Concrete Piles Ltd.	—
Stelmo Ltd.	lxxix
Stent Precast Concrete Ltd.	i
Stic B Paint Sales Ltd.	lxi
Sussex Court Ballast Co. Ltd.	lxxxix
Sykes, Wm. T., & Co. Ltd.	xxiv
Symonds Developments Ltd.	lxiii
Stuart's Granolithic Co. Ltd.	lviii
Tarmac Viscolite Ltd.	xxxi
Taylor Woodrow Constructions Ltd.	—
Technical Paint Services	xx
Tenton Bar Co. Ltd.	—
Terrenearch Ltd.	xxviii
Thames Plywood Manufacturers Ltd.	—
Trent Gravels Ltd.	lxiv
Trentham Gravel Co. Ltd.	—
Truvex Concrete	—
Tunnel Portland Cement Co. Ltd.	lxxxi
Volkmann, O. A.	lx
Walker Wright & Co. Ltd.	xlviii
Welbeck Trading Co. Ltd.	—
Whitwood, Joseph, & Co. Ltd.	—
Whitehead Iron & Steel Co. Ltd.	xxxi, xxvii
Whitley Moran & Co. Ltd.	xi
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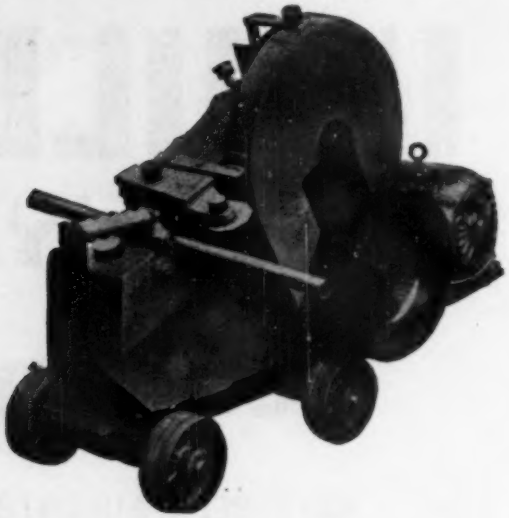

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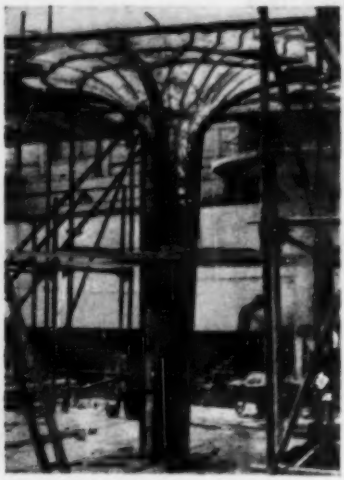

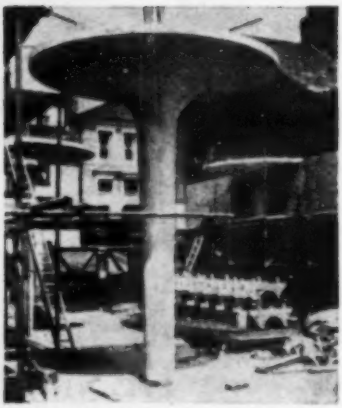

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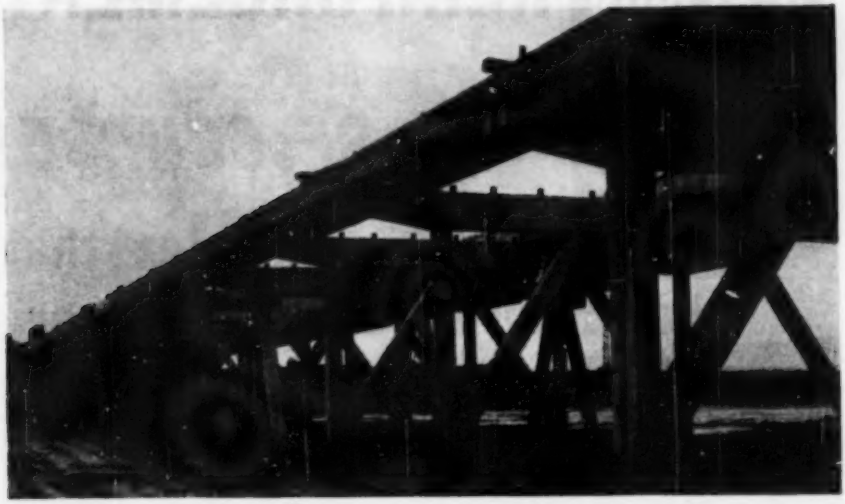

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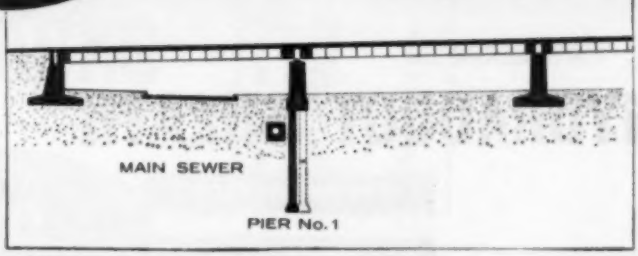

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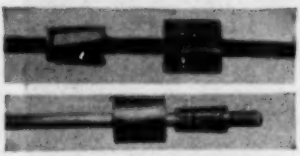

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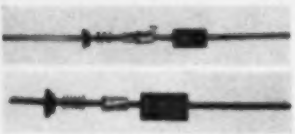

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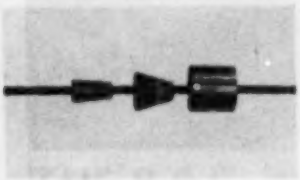

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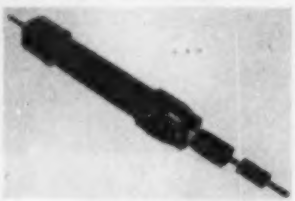

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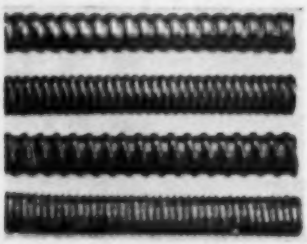

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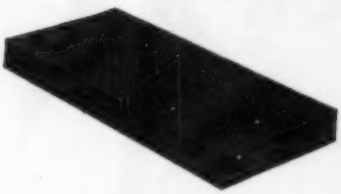

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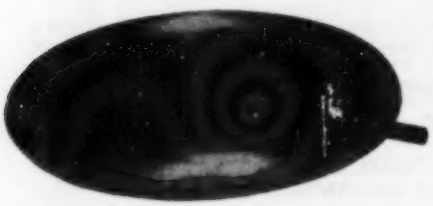

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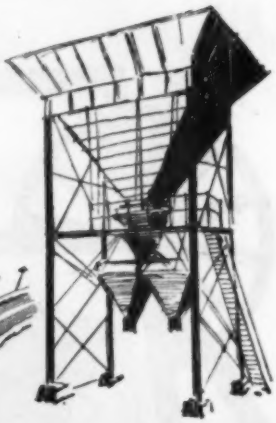

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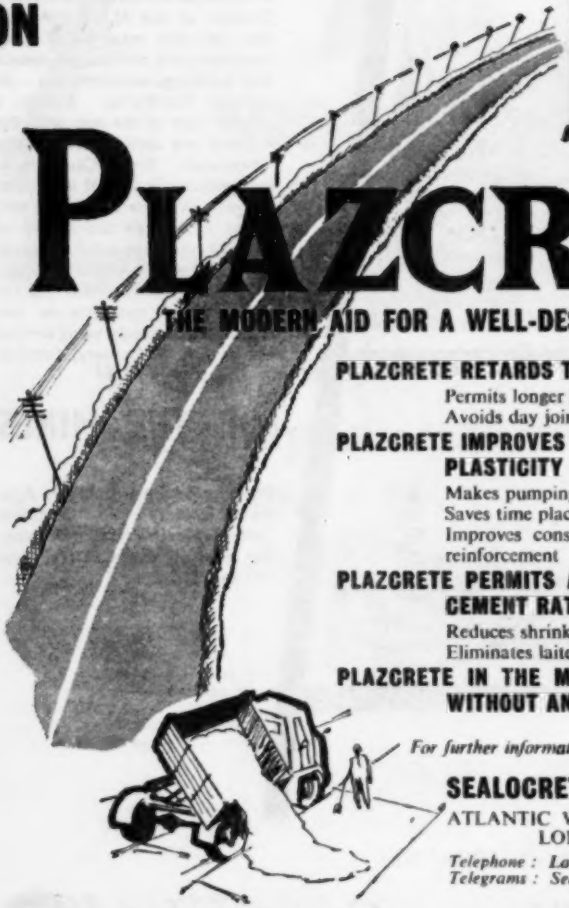

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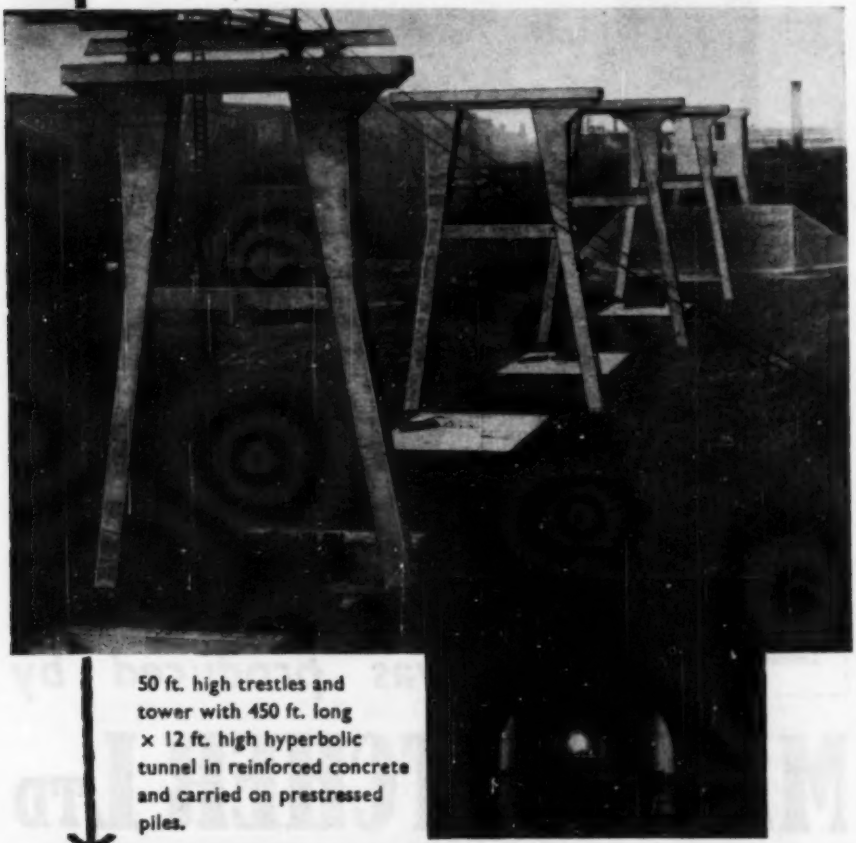

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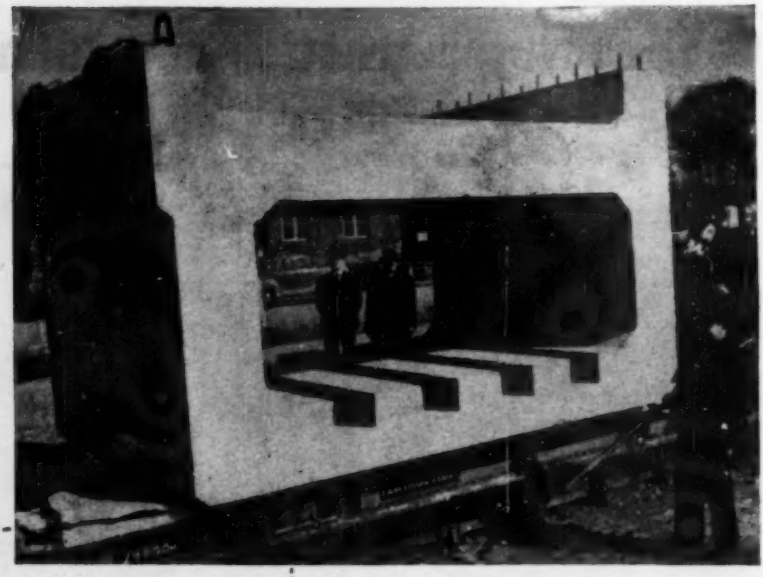

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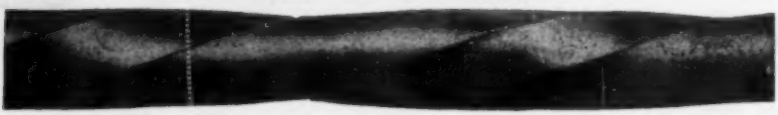

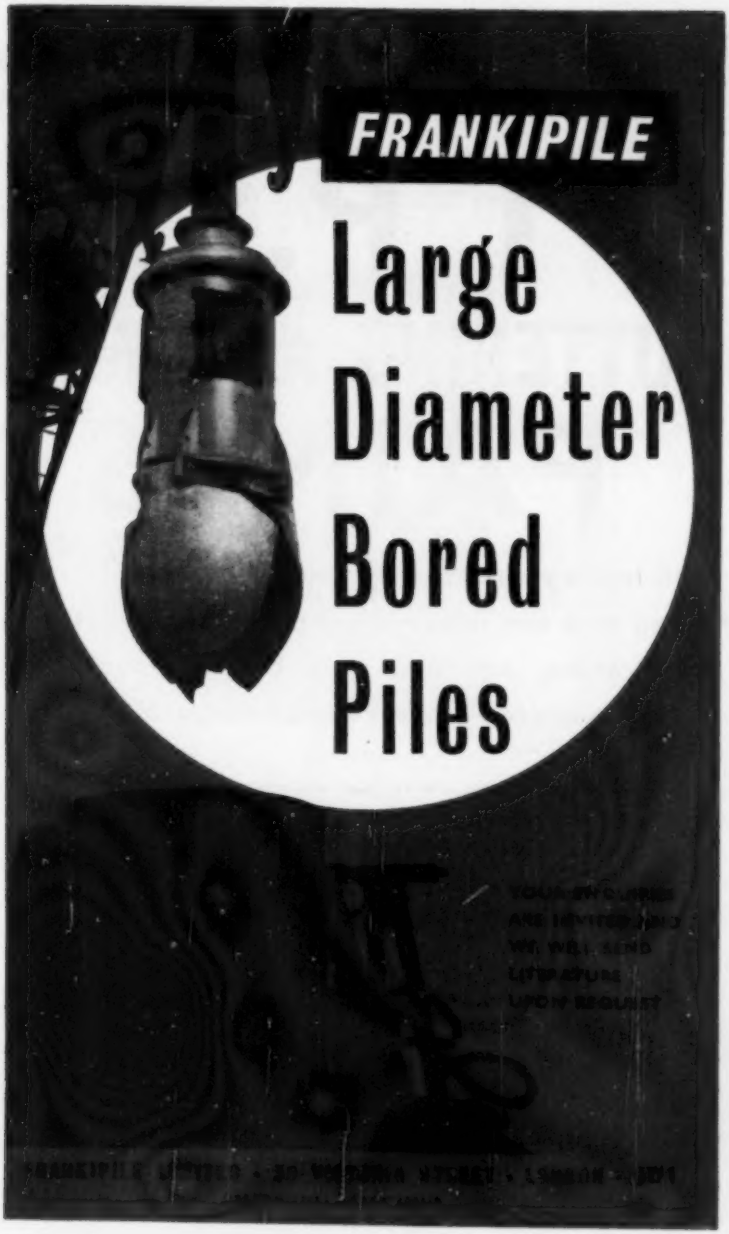
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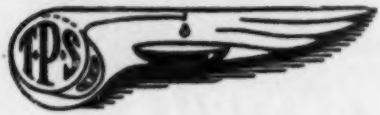

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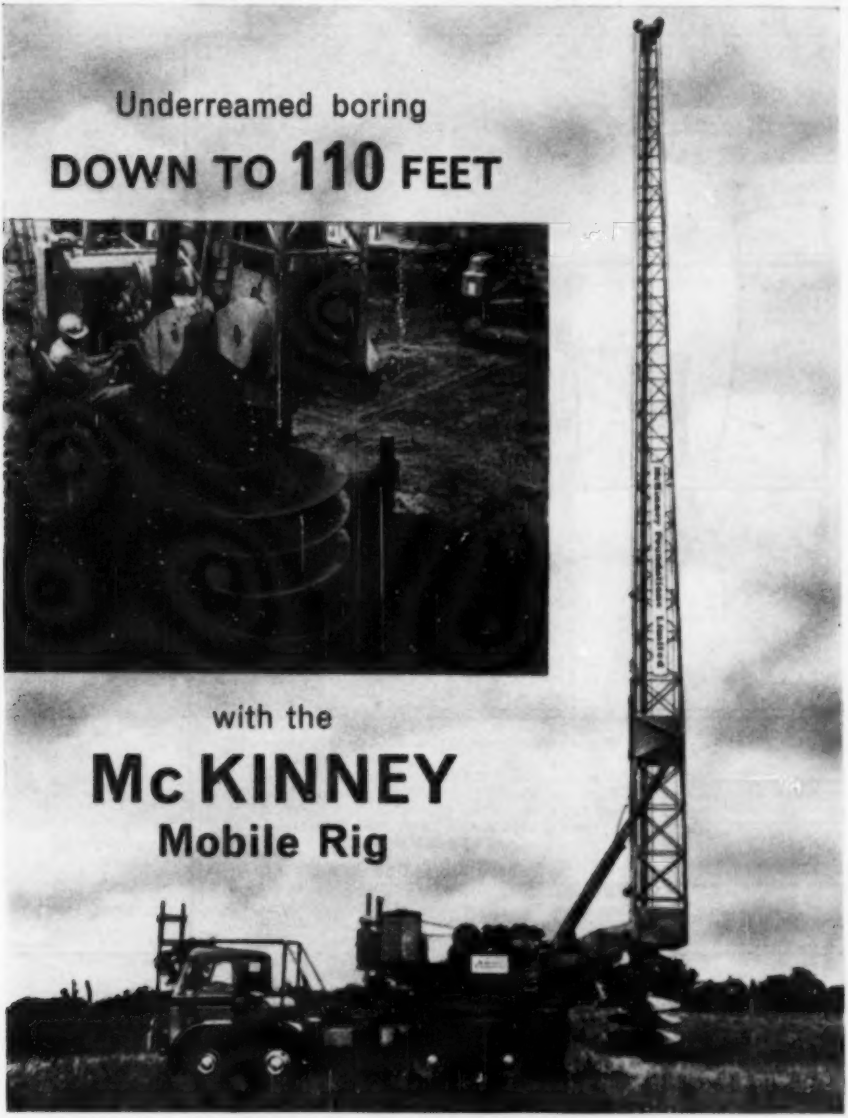

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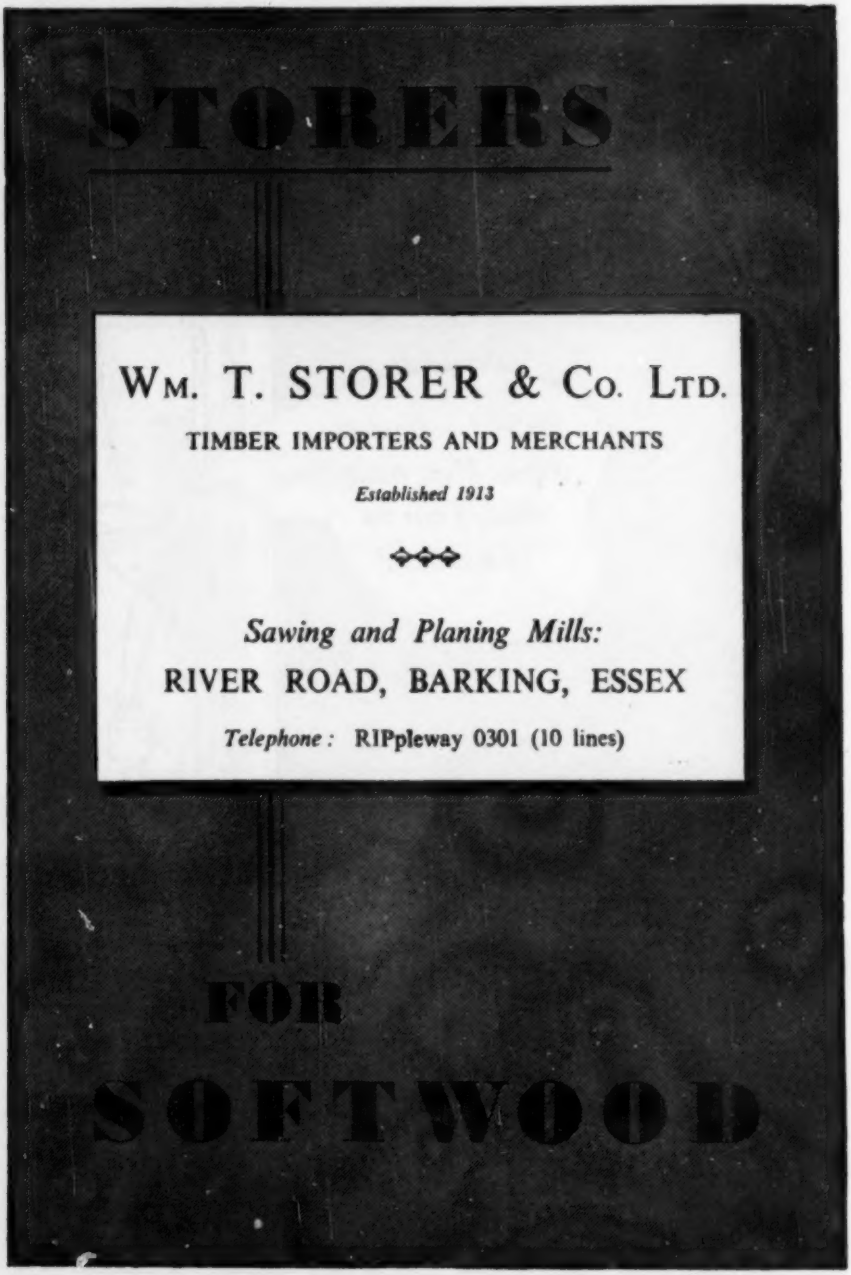
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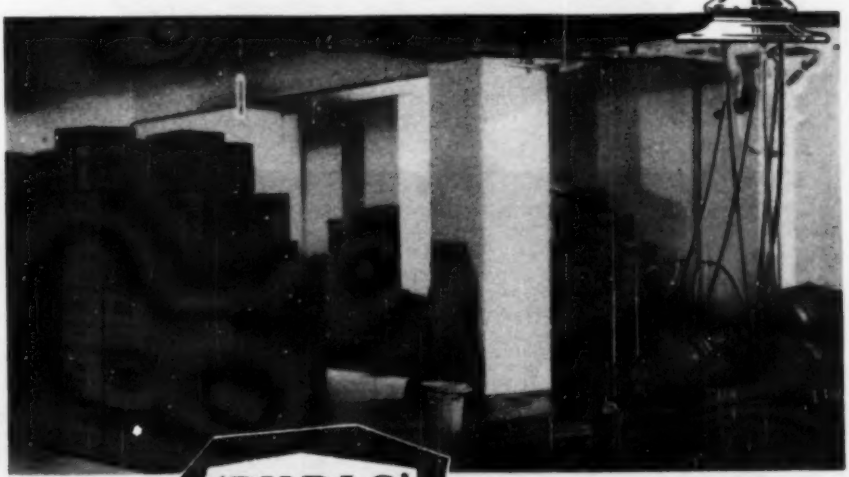
Beer Cellar of the 'HARP,' Parson's Mead, Croydon. By courtesy of Page and Overton's Brewery Ltd.



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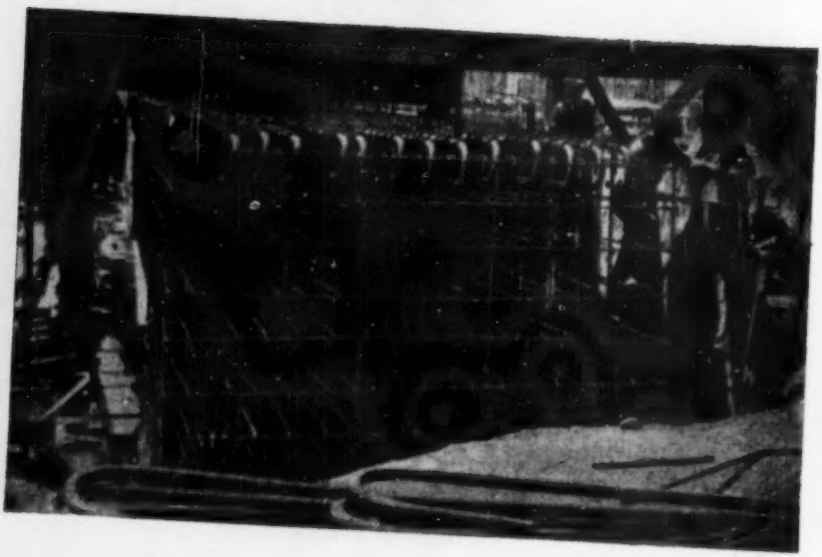

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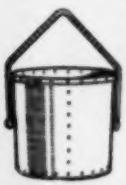

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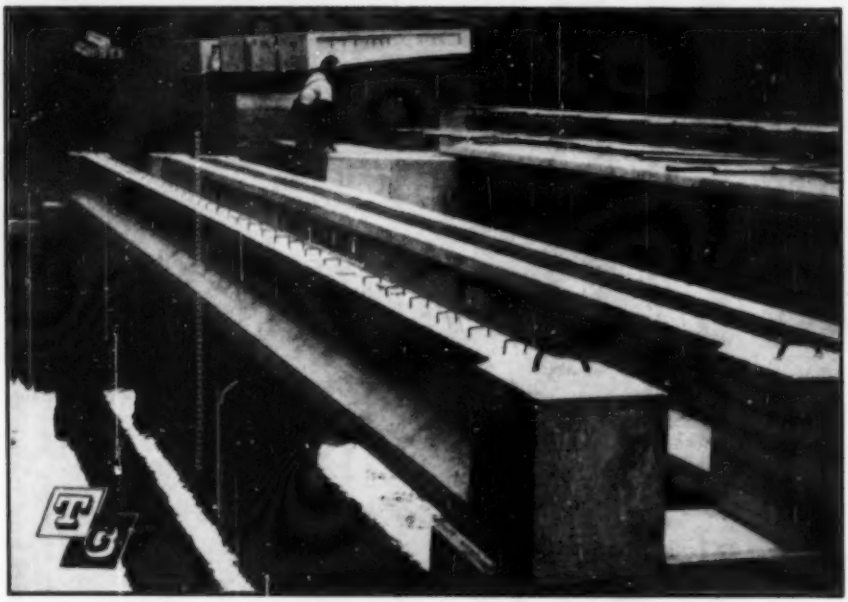

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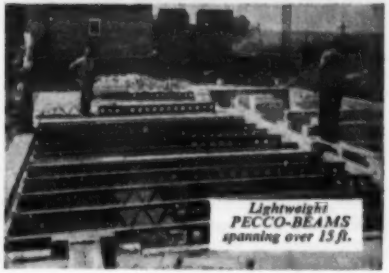

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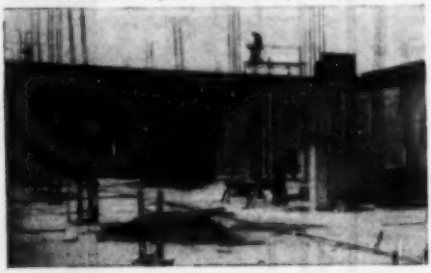

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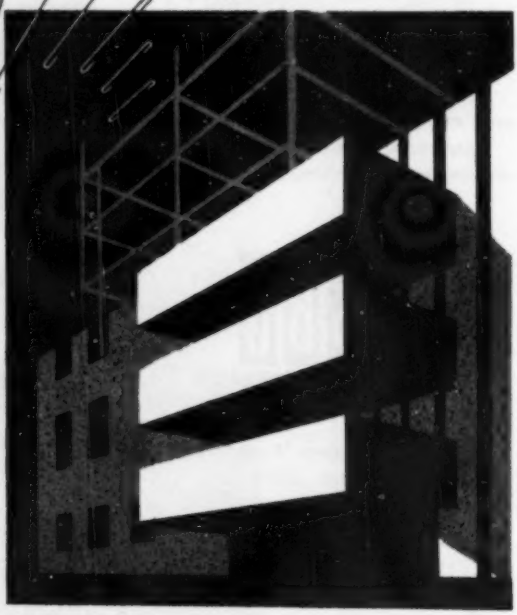

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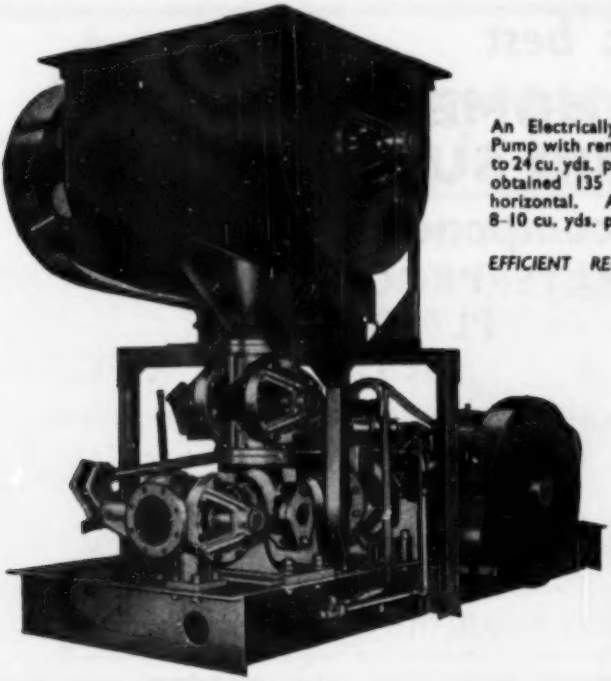

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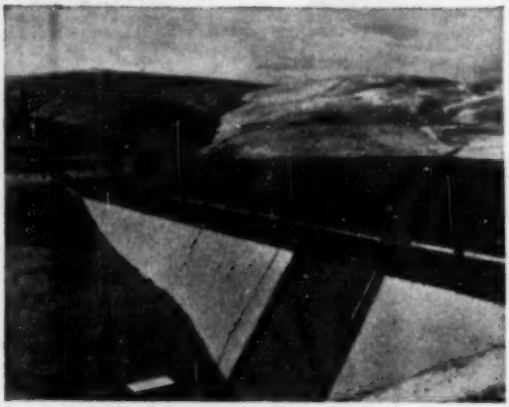
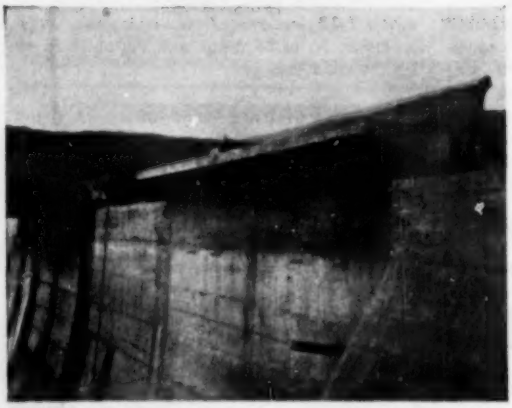


IN BRITAIN Darex AEA was used throughout in the concrete construction of the Glendevon Dam



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Darex AEA is the original air entraining agent that has proved itself throughout the world for Dam Construction, Structural Concrete, Roads and Aerodromes, Bridges, Maritime Works and Concrete products. America, Belgium, Canada, Norway, Portugal, Sweden, Switzerland and many other countries acclaim Darex AEA as the first and foremost air entraining agent.

TECHNICAL SERVICE may be obtained from the sole U.K. distributors EVODE OF STAFFORD

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ask for Darex AEA

as the fifth ingredient for concrete

DAREX AEA, a ready-to-use, dark brown liquid is an aqueous solution of triethanolamine salts of a sulphonated hydrocarbon. For smoother flow and finer finishing, it lubricates the concrete mix with millions of tiny air bubbles. It also contains a catalyst which promotes more rapid and complete hydration of portland cement. Not a resin or by-product, DAREX AEA is a chemical formulation manufactured to insure predictable performance. It is harmless and neutral and requires no pre-mixing with corrosive chemicals or water.
DAREX AEA is ideally suitable for ready mixed concrete, structural concrete, mass concrete construction, concrete road construction, repairs and patching and concrete products.

- Air bubbles guard against frost damage and cracking even with repeated freezing and thawing.
- Makes concrete resistant to saline solutions, de-icing salts, sulphate and sea waters.
- Less mixing water can be used with no loss in slump.
- Improves placeability.
- Air bubbles lubricate and plasticize the mix.
- Minimizes bleeding, green shrinkage and segregation.
- Catalyst promotes faster and more thorough hydration, thus bringing out more of inherent strength of portland cement; assures greater strength in both lean and rich mixes than non-catalysed air entraining agents.
- Will not entrain excessive air.
- Particularly effective with slag, lightweight or manufactured aggregates that tend to produce harsh concrete.
- Permits use of natural sand deficient in fines.

Reduce water with WRDA

WRDA, an aqueous solution of highly purified metallic salts of lignin sulphonic acid, is a ready-to-use, non-setting dark brown, mobile liquid. It acts as a dispersing and plasticizing agent—makes possible easily placeable mixes with up to 20% less water content and a corresponding increase in strength and durability. It contains a catalyst which counteracts the normal hydration-retarding effect of dispersing admixtures; and a preservative. It is compatible with all known air entraining agents. WRDA does not require agitation, it comes ready-to-use.

- WRDA reduces natural interparticle attraction of cement grains and makes the mix more workable and placeable with less water.
- Gives low slumps without loss of placeability.
- Makes concrete flow cleanly around reinforcing steel and respond quickly to vibration, spading and compacting.
- Improves water retention and internal cohesiveness of the plastic mix.
- Cuts green and drying shrinkage.
- Reduces bleeding, honeycombing, water gain voids under aggregate and other segregation.
- Allows concrete floors to be finished sooner.
- Increases compressive strength up to 25%; also increases flexural and tensile strengths.
- The action on cement is physical. It does not change the chemical properties.
- Improves appearance of concrete by sharply reducing spalling, leaching and sand streaking.

AIR METERS AND DISPENSERS ARE READILY AVAILABLE

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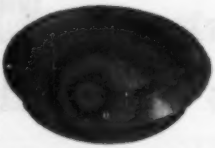
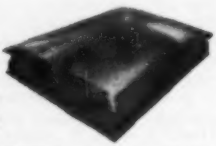
maximum light minimum cost

LITEX roof lights give a very high transmission of light—up to 92%—and are easily and quickly handled and fitted. A full range of standard form metal ventilators, curbs and liners are available for use with LITEX roof lights.



LITEX spherical roof lights

Litex LAMINATED FIBREGLASS roof lights



LITEX rectangular roof lights

LITEX roof lights with their glass fibre reinforced construction can be used with advantage in many places where conventional lights would be impracticable or too costly. Virtually unbreakable, heavy packing is unnecessary allowing quick and easy handling on site.

LITEX roof lights are ideal for use in schools, hospitals and other buildings where absolute safety is required—they will not shatter and cannot support combustion. The plastic will not creep, nor become brittle with age or very hot temperatures.

LITEX roof lights are not attacked by smoke or fumes from most industrial processes, and may safely be used in factory and workshop buildings.

LITEX lantern lights

For further information about LITEX roof lights please write to

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One of the biggest advantages of the high-speed Winget Turbomixer is its ability to give the highest degree of activation *in seconds!* That means designed strength faster and cheaper—an unbeatable profit-making feature for any pre-cast products manufacturer. That's why you'll find Turbomixers installed in plants all over the world. Being completely self-contained, also, the mixer can generally be installed without alteration to existing structures. There are mixers of all sizes to meet all outputs. Send for full details without delay.

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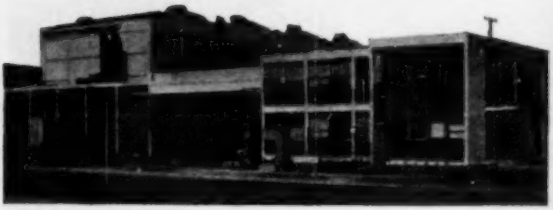

The new British Visqueen factory at Stevenage was built to produce the largest output of polythene sheeting in Europe; and it is here that the Visqueen Building sheet is also manufactured to supply an increasing number of applications for it in the building industry.

To render the concrete factory floors hard, dustless, waterproof and able to withstand exceptionally heavy wear and tear, British Visqueen chose Lillington's No. 1 Metallic Liquid.

For nearly fifty years, architects have specified Lillington's No. 1 Metallic Liquid, the scientifically prepared admixture that renders concrete completely waterproof and dustless and accelerates setting time.

No. 1 Metallic Liquid is a necessity for waterproofing cement renderings to walls and basements, in mass concrete retaining walls, foundations, flat roofs and tanks. You can rely on it to give complete satisfaction because IT IS THE ONLY PROOFER SOLD UNDER GUARANTEE.

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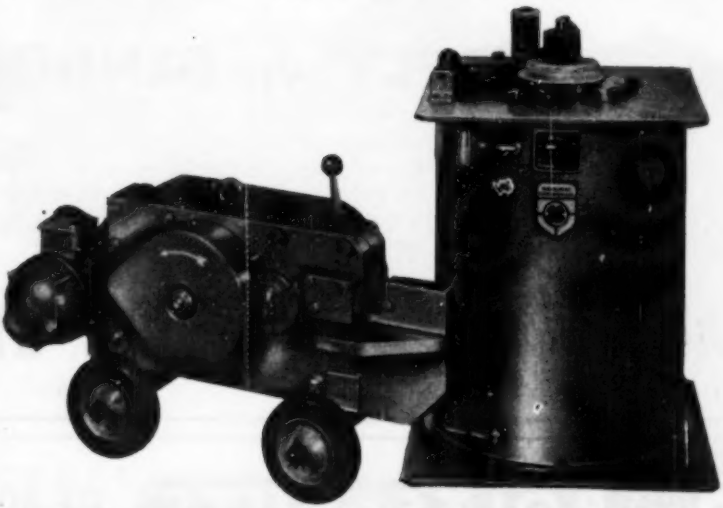

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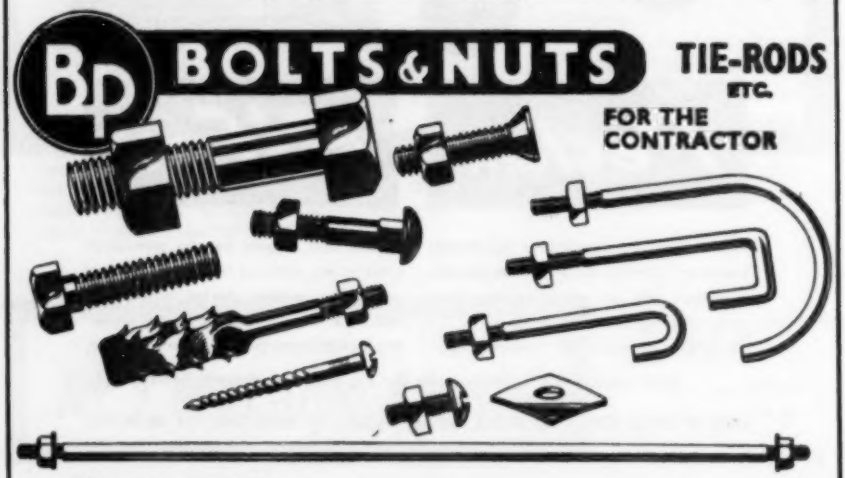

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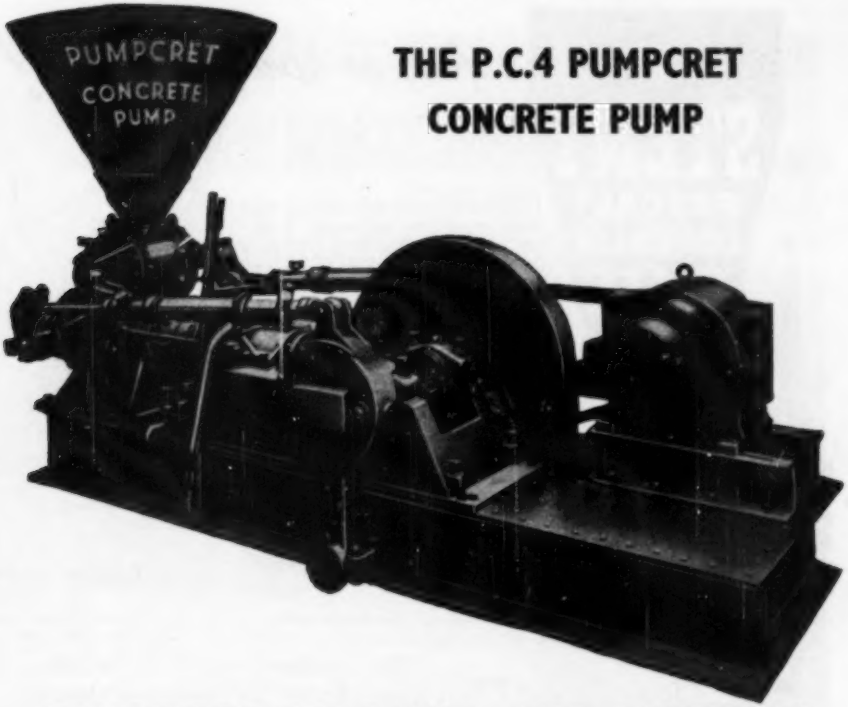

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Horizontal, approx.	1500 ft.	1250 ft.
or Vertical, approx.	135 ft.	125 ft.
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- The model illustrated here operates at 9,000 to 15,000 Vibrations per minute from low flexible shaft speeds of 4,500 to 7,500 R.P.M.
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This unique machine, with quick-change additional tools, can also be used for: SURFACING, WET-RUBBING CONCRETE, GRINDING, DISC SANDING, AND DRILLING (up to 1¼" in Concrete, 1" in Steel, and 2" in Wood).

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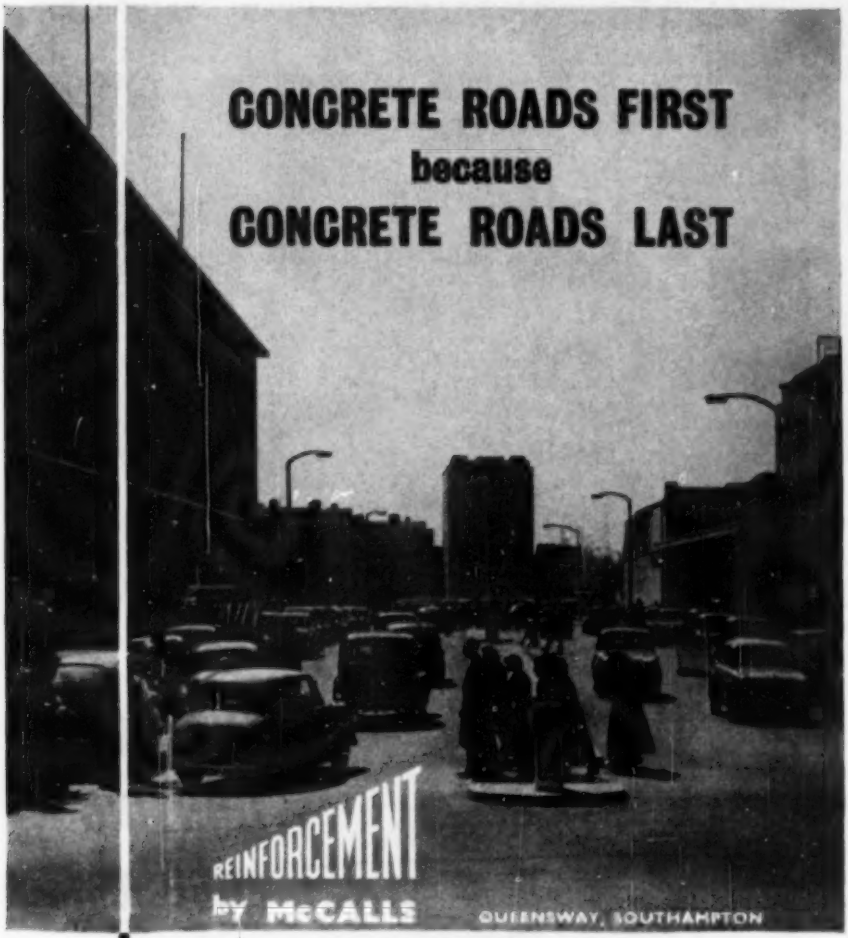

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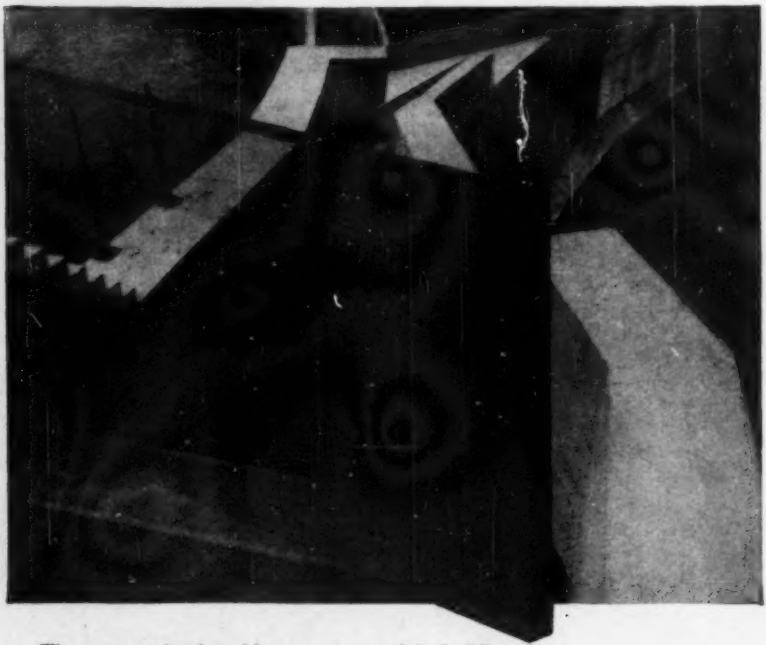

The constructional problems encountered in building this reservoir are similar to those found in a wide variety of water-retaining structures. It is imperative that joints between supporting columns and the floors are resistant to seepage and yet allow for any expansion or contraction between them.

Expandite products solved this problem of 'joints-that-move'. PVC Hydrofoil waterstops were cast in the concrete; the joints were then packed with strips of 'Flexcell', a non-extruding joint filler (specially manufactured for Expandite by Celotex Ltd.) and then top-sealed with 'Pli-astic', a hot-poured rubber-bitumen joint sealing compound and, in certain cases, with 'Plastijoint' flexible, waterproof sealer.

Movement was allowed for, but, because all these products are sufficiently flexible to allow for movement and yet retain their original shape on contraction, no seepage will occur.

Problems of 'joints-that-move' take many forms, but Expandite Limited manufacture products to solve them all. Architects, Builders and Civil Engineers are offered advice — without any obligation — on all sealing and jointing matters they may wish to put before the Expandite Technical Service Department.

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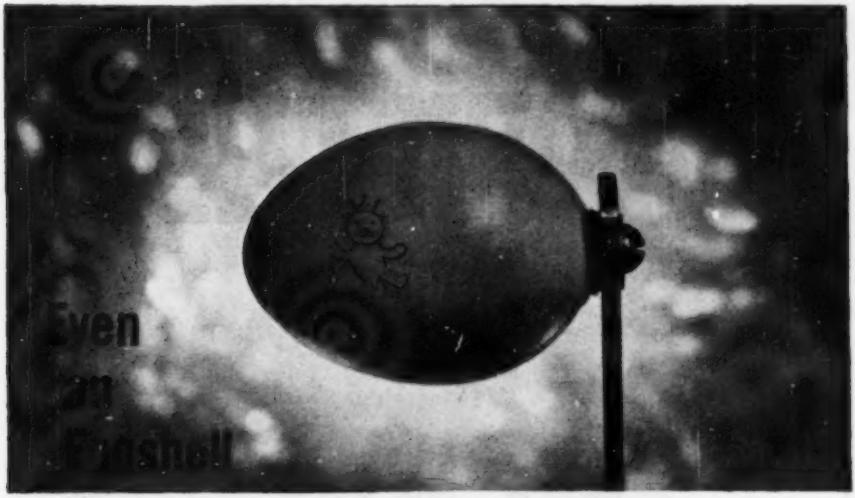

can be Screw-fixed with a RAWLPLUG FIXING DEVICE!

If you live to a hundred, you are unlikely to want to do much screw-fixing of eggshells! Yet this feat—possible only with a Rawlplug Fixing Device (a Rawlnut)—does serve to highlight the astonishing effectiveness of these Devices in making 'difficult' and even 'impossible' fixings simple and straightforward. Whatever the screw or bolt fixing job, you'll save time, money and temper by using the appropriate Rawlplug Fixing Device.

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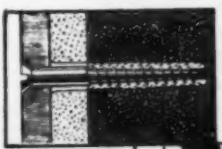
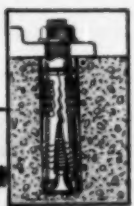
The famous Rawlplug makes firm screw fixings in masonry in a mere fraction of the time taken by any other method. For all screw sizes up to ½" diam. Coach screws.



RAWLBOLTS

For light or heavy bolting jobs. A dry fixing of enormous strength—no cold chiselling, no waiting for cement to harden. In all bolt diameters up to 1".



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The amazing Rawlnut forms its own 'rivet head' behind the material when screwed up from the front. Shakeproof and waterproof, it has many valuable uses in both building and manufacture.

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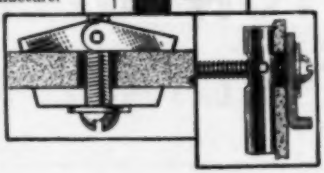
For making firm fixings to such thin and structurally weak materials as plasterboard, ceilings etc. The wings of the device spring apart behind the material and spread the load over a wide area.



GRAVITY TOGGLES

Passed through a hole in hollow material, the long member falls into a vertical position by gravity, and is then drawn against the back of the material by screwing from the front.

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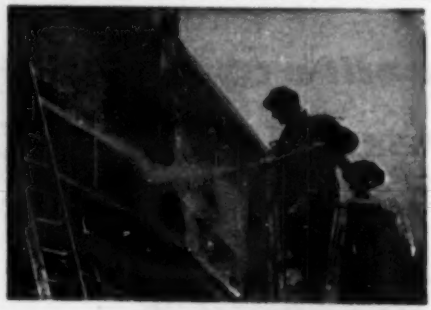

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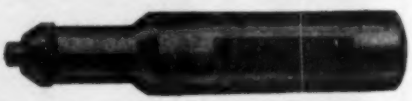

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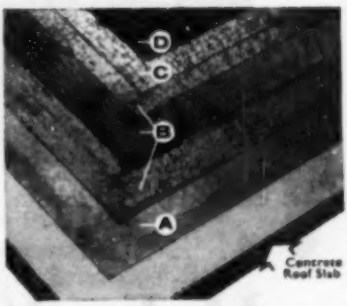

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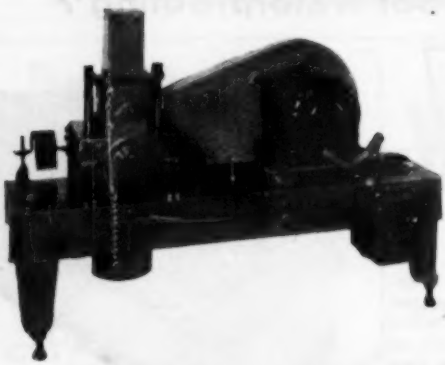

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Vibro

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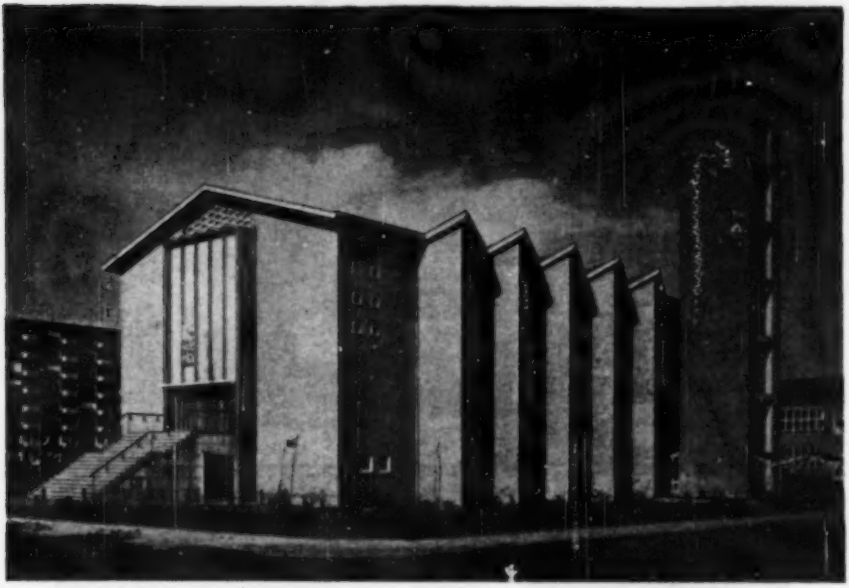

This new church at Vlaardingen near Rotterdam is founded on fifty-nine 17-in. diameter Vibro piles 81 ft. long. The working load per pile is 68 tons, and 19-in. and 24-in. diameter shoes were used to develop the appropriate bearing capacity.

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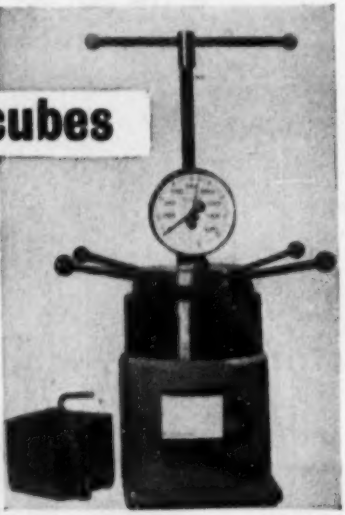
Testing to 16,000 lb. per sq. in. to B.S. 1881.



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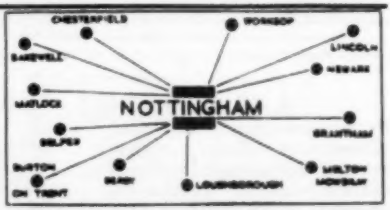

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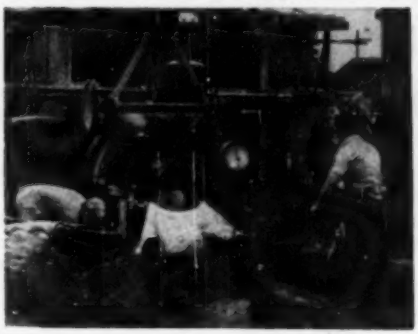

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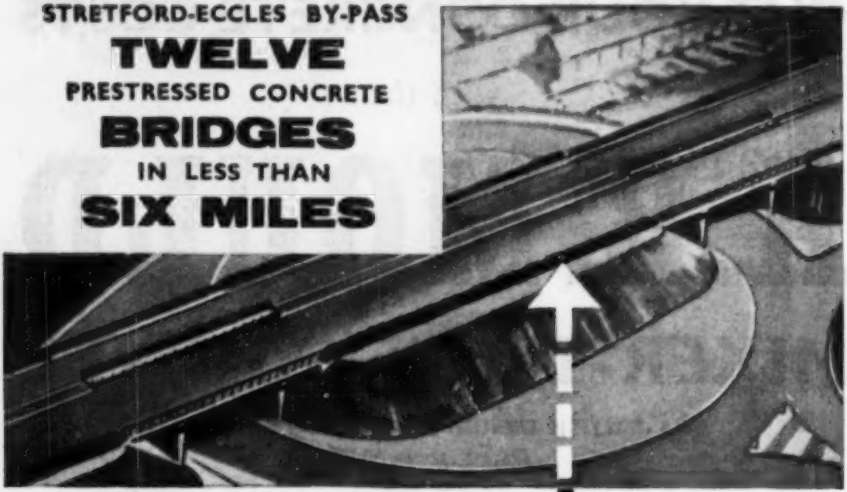

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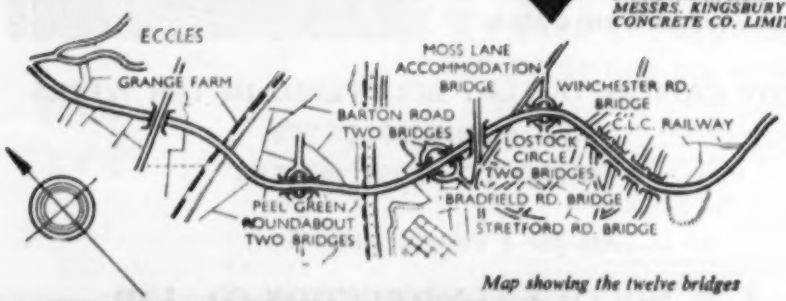

Map showing the twelve bridges

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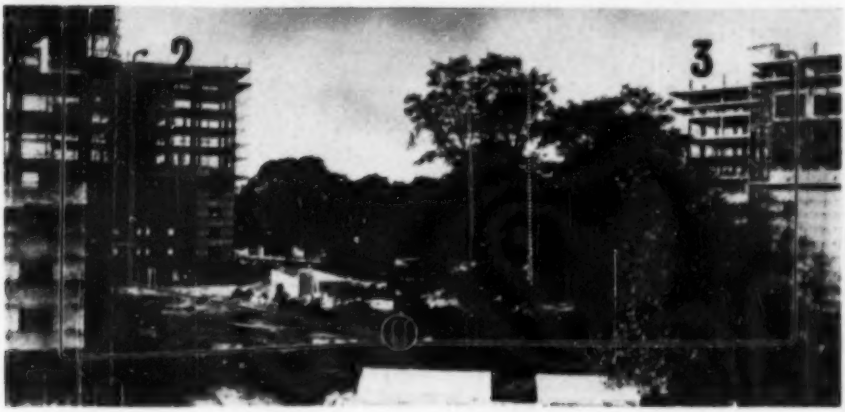

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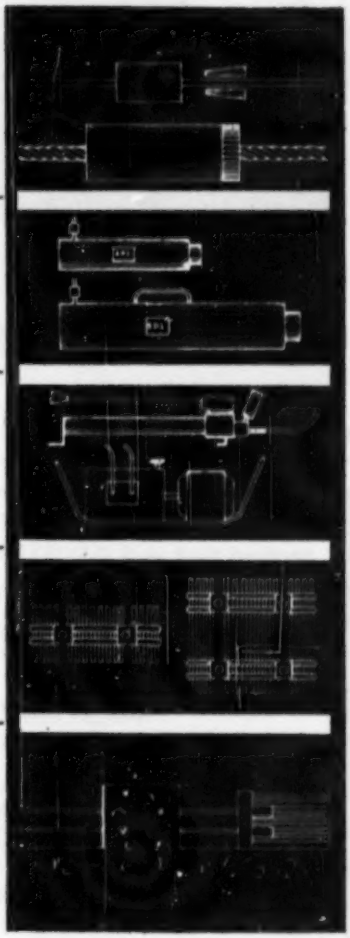

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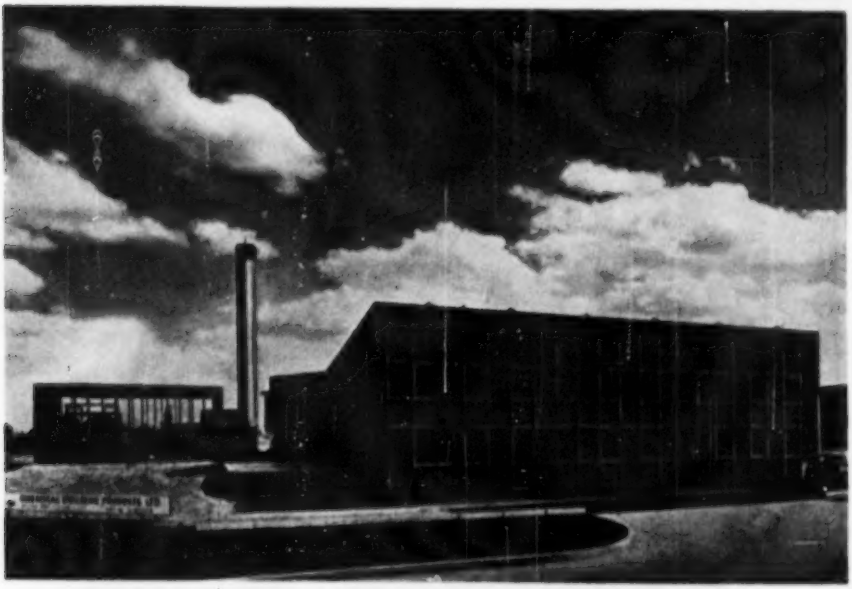

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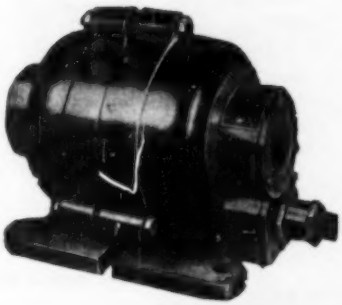

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume LV, No. 5. LONDON, MAY, 1960.

EDITORIAL NOTES

The Earnings of Engineers.

SOME interesting facts concerning the remuneration of British engineers, whether salaried or fee-earning, are given in a report,* published recently, by a Royal Commission. Although the Commission was concerned particularly with the question of whether or not medical and dental practitioners were underpaid, comparisons are made of the earnings of accountants, actuaries, architects, barristers, engineers, solicitors, surveyors, and teachers at universities. It is seen that, at all ages from thirty to sixty years, the median earnings of engineers are less than that of members of any of the other professions selected with the exception of architects. Nevertheless, the earnings of engineers appear to increase steadily throughout their working life, whereas in most of the other professions the annual earnings reach a peak around the age of fifty and decline thereafter; the decline is particularly noticeable among general dental practitioners. Exceptions to this decline are medical consultants, solicitors in England and Wales, and particularly actuaries. (The median earnings are obtained by setting down in one column the incomes, in the order of magnitude, of the members in one group; the median is that amount which is in the middle of the column, that is, there are as many entries above as below. The earnings of an individual are considered to be the income agreed with the income tax authorities.)

According to this survey, an engineer at the age of thirty can expect to obtain, on the average, from £800 to £1200 per annum; at the age of sixty, the average range is from £1200 to £2400. One quarter of the members of the engineering profession receive less than the lower limits of each of these ranges and one quarter receive more than the upper limits; in fact, the annual remuneration of some engineers exceeds £6000 and in a few cases £10,000. It is, however, the average range that matters, since those earning amounts below the range are either in the wrong profession or are of such ability that they would not be successful in any profession, and those in the high-income groups are undoubtedly men who would be a success in any calling. The corresponding average earnings of surveyors are about the same as those of engineers at thirty but range up to £3500 at sixty, and a higher proportion earn more than £6000 and £10,000, even

* Report of the Royal Commission on Doctors' and Dentists' Remuneration; 1957 to 1960. (London: H.M.S.O. Price 15s.)

if surveyors in public service are included. Actuaries and barristers show up best in the high-income group, although in the average cases barristers are well down the scale except in early middle-age. In general, university teachers rank better than engineers although there are few, if any, enjoying annual incomes exceeding £6000.

It is interesting to compare the earnings of engineers with the remuneration of other professional men abroad as given in the Report. In general, engineers, accountants and architects, in this ascending order, are less well paid than are doctors, dentists and lawyers, although the relative positions in the scale vary in different countries. In the U.S.A., Canada, and Australia an engineer is likely to receive less than an architect, but in Belgium, although an architect may earn more than an accountant, an engineer may well earn more than an architect. According to official statistics relating to the earnings of professional men in Canada and issued in 1958, dentists and accountants may earn less than £4000 in a year, lawyers and doctors up to £5000, but consulting engineers earn, on the average, more than £5000.

The total earnings of an engineer or architect, in Britain, from the age of thirty to sixty-five are likely to be lower than those of a man engaged in any of the other professions considered. The plums, represented by a total income of more than double that of an average engineer, fall to medical consultants, hospital doctors, actuaries and, to a slightly less extent, barristers. By this basis of comparison, surveyors and university teachers do little better on the average than do engineers. Allied to this comparison is the fact that the cost of training for engineering may be less than that for some other professions, but the capital investment required to be made by a consulting engineer in setting-up his business is in general much more than that of other professional men.

For a survey of this kind to have real value, it is important for the data to be collected from the entire body of practitioners in each profession; since this is generally impracticable, except perhaps in the case of civil servants and those in nationalised industries, there must always be some doubt as to the representativeness of a partial survey. In the case of engineers, the Commission obtained its data by circularising 9900 members of the Institutions of Civil, Mechanical and Electrical Engineers, a number which represents less than a fifth of the membership of these Institutions. Because these Institutions may not deal with matters concerning the remuneration of their members, the survey was conducted with the co-operation of, and partly at the expense of, the Engineers' Guild. About 74 per cent. of the members, who were asked to give particulars, did so. Since different methods of collecting data were adopted in each profession considered, the results may not in all cases be comparable; for instance, in the case of architects the survey covered about a fifth of the membership of the Royal Institute of British Architects; about one tenth of the practising accountants were applied to, whereas all professional actuaries were approached and nearly all replied; a third of the number of chartered surveyors, excluding those in official positions, were included.

If the data so laboriously collected and collated by the Commission depicts a true picture of the position, it is obvious that higher remuneration is essential in the engineering profession if men of the right calibre are to be attracted to this profession. The Commission's conclusion is that, in general, doctors and dentists

are underpaid; but since the data show that the remuneration in these professions is considerably higher than in most professions, especially engineering and architecture, surely the case for improving the financial status of engineers and architects is equally good, if not stronger. So far as engineers are concerned, it is understood that the Engineers' Guild is endeavouring to effect such an improvement. On the other hand, it can be argued that the majority of engineers are engaged in drawing offices and elsewhere in positions for which the present remuneration is commensurate with the moderate requirements of technical experience and responsibility, and that, either in senior positions in employment or in private practice, there is ample scope for able engineers to obtain remuneration on a reasonably high scale.

An Honour for Professor P. L. Nervi.

PROFESSOR PIER LUIGI NERVI is recipient for 1960 of the Royal Gold Medal for Architecture, which is an award made annually by Her Majesty the Queen on the recommendation of the Royal Institute of British Architects.

As part of the arrangements connected with the award an exhibition of photographs and drawings of the work of Professor Nervi was held in London in April. The exhibition was organised jointly by the Royal Institute of British Architects and the Cement and Concrete Association. Some examples of Professor Nervi's work in reinforced concrete are given on pages 188 and 189 of this number; these examples are of works not illustrated previously in this journal (see November 1955, pages 387 to 393, and January 1956, pages 221 and 222). Although much of his work is in Italy, Professor Nervi has designed structures in other countries, in some cases jointly with other engineers and architects. A notable structure in France is the UNESCO headquarters in Paris (see this journal for December 1959), and his designs will be adopted for the proposed Washington Bridge bus station in New York and for the cathedral and abbey of New Norcia, near Perth, Australia. A feature of the latter structure will be intersecting parabolic reinforced concrete arches 100 ft. high and of 100-ft. span.

The Yield-line Method.

THE British Standard Code of Practice for Reinforced Concrete in Buildings (1957) recommends three methods on which the design of slabs spanning in two directions may be based. One of these methods is the yield-line method which is not used as much in this country as are methods based on the analysis of thin flat plates. Among the first exponents of the yield-line theory were C. Bach, O. Graf, and E. Mörsch in the early decades of this century, and later Aa. Ingerslev. The work of K. W. Johansen, published from 1931 onwards, is accepted to-day as the basis of most applications of the theory. H. Craemer introduced the subject in this journal in August 1950.*

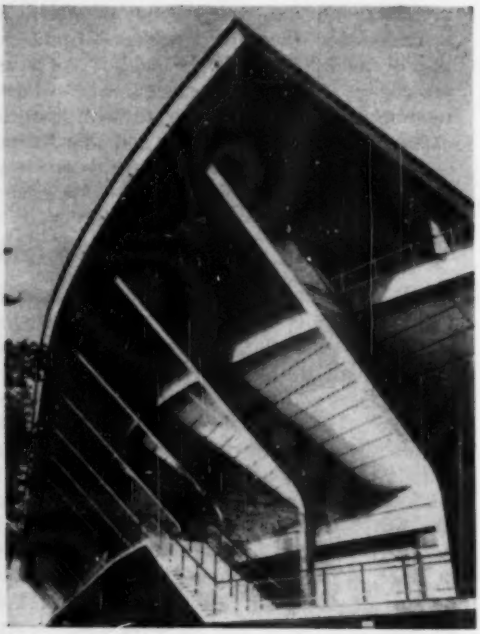
An article giving the principles and practical application of the method is given in this journal for March 1960, and is continued in this number. In subsequent numbers we hope to give further articles on this subject by various writers.

* An annotated bibliography of the yield-line method from 1898 to 1958 is published by the Cement and Concrete Association and is obtainable free from the Association at 52 Grosvenor Gardens, London, S.W.1.

Some Recent Works of Professor P. L. Nervi.

(*See page 187.*)

Fig. 1.—The Pirelli Building, Milan.



The undersides of the precast slabs forming the seating and extending between the inclined frames are smooth and contrast with the shutter-board marks on the faces of the cast-insitu frames.

Fig. 2.—Rear of Grandstand at the Flaminio Stadium, Rome.

Fig. 3.—The Grandstand at the Flaminio Stadium, Rome.

Fig. 4.—Sports Palace at Rome. (Palazzetto Dello Sport.)

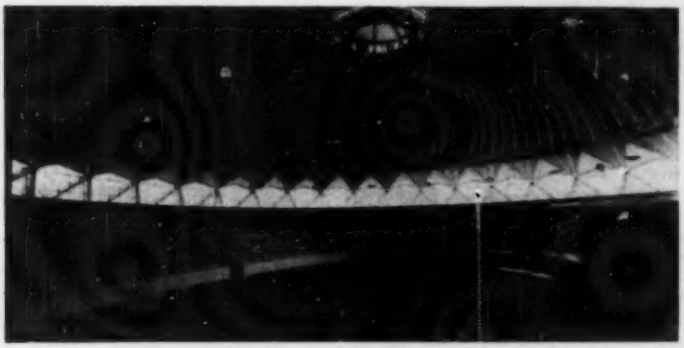


Fig. 5.—Interior of Sports Palace, Rome.

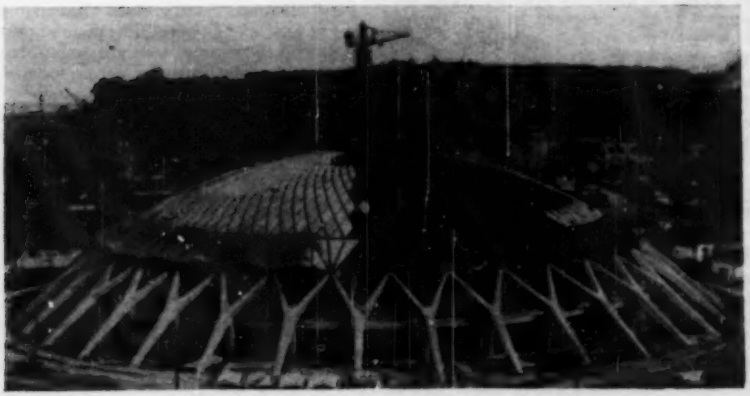


Fig. 6.—Sports Palace, Rome, in Course of Construction.

Book Reviews.

"Third Congress of the Fédération Internationale de la Précontrainte: Berlin 1958." Obtainable from Concrete Publications Ltd. Price by post (for two volumes) £13; in Canada and U.S.A., 50 dollars.

THE first of the two volumes contains over seven hundred pages and gives in full the text of the sixty-two papers presented at the Third Congress of the International Federation of Prestressed Concrete which was held in Berlin in 1958. The Congress comprised three sessions which dealt respectively with (I) developments of methods of design; (II) progress in methods of prestressing as applied on the site, and with special reference to grouting, anchorages, reduction of friction, and safety precautions; and (III) progress in the manufacture of prestressed units made in factories, and in the use and erection of such units. General reports summarising the contents of the papers presented in each session are also included.

The second volume, containing about two hundred pages, records primarily the discussions amounting to seventy-three contributions, nearly half of which are related to the subject-matter of session II. This volume also contains the general reporter's introductions and summing-up of the discussion in each session.

The papers and discussions are printed in the language in which the contribution was made, namely, German, English, French, or Spanish, but a summary of each paper is given in each of these languages. It is interesting to note that of the papers presented those from Germany outnumbered those from any other country, with papers from Great Britain in second place. In the discussions, more contributions came from Great Britain than from any other country.

"Concrete and Soil-Cement Roads." By W. P. Andrews. (London: Contractor's Record, Ltd. 1960. 132 pages. Price 30s.)

THIS well-illustrated book deals with the subject of the design and construction of concrete roads and allied construction in as much detail as can be expected in comparatively few pages. The principal contents include the preparation of the subgrade, the materials used, the making of concrete, curing concrete (there is no entry under this heading in the index), design of roads, reinforcement, joints, construction by manual and mechanised methods, tests, maintenance, failures, repairs, roads for special traffic such as military tanks, prestressed concrete roads, roads on peat, farm roads, kerbs and haunches, coloured concrete, cement-bound macadam, cement-grouted and cement-soil roads (five chapters are devoted to the latter type), and lean-concrete bases. Many of the works dealt with are British, but comparable practices in other countries are described. Reference is made to the work of the British Standards Institute, the Cement and Concrete Association, and the Road Research Laboratory.

"An Introduction to the Dynamics of Framed Structures." By G. L. Rogers. (London: Chapman & Hall, Ltd. 1958. Price 82s.)

ALTHOUGH there are many books on the statical design of structures few of them deal comprehensively with the effects of dynamic forces. This U.S.A. book deals with the stresses in structures subjected to dynamical disturbance caused by earthquakes, blast (particularly from atomic explosions), unbalanced machinery, and rolling loads. The types of structure considered include frames having one or more degrees of freedom, truly rigid frames, continuous beams, and some forms of floors. The constructional material does not enter primarily into the problem except in so far as the member is elastic, inelastic, or visco-elastic.

The treatment is mainly theoretical and is given as a basis for further development rather than an endeavour to deal with the whole range of principles and practice. The work is therefore primarily mathematical, and processes, for example, matrices and orthogonal functions, are used which seem most suitable to the problem. For a reader whose mathematics does not extend beyond elementary calculus, a discussion of the methods of calculation is included which, as the author hopes, should make it unnecessary to refer to treatises on mathematics.

Stresses and Factors of Safety in Prestressed Concrete.

By P. W. ABELES, D.Sc., M.I.Struct.E.

THE working stresses recommended in the new British Standard Code[1] for prestressed concrete are compared with the corresponding recommendations in the U.S.A.[2] and Germany[3] in this journal for February 1960. In the following are given some further comments on these stresses and views on the factors of safety necessary in prestressed concrete.

Permissible Stresses.

According to the German recommendations, when partial prestressing is employed for road bridges, the permissible tensile stresses in the concrete in a member with the tensile zone at the bottom are less than recommended in the general case and are from 426 lb. per square inch to 639 lb. per square inch, depending on the grade of concrete. When the tensile zone is at the top, these stresses apply only if the member is well protected.

In 1959, a tentative code was issued by the Prestressed Concrete Institute (U.S.A.).[4] There is little difference between this code and the U.S.A. recommendations except for the inclusion of recommendations for partial prestressing. The same conditions as for ordinary prestressed concrete are recommended for ultimate-load design, but no limits are placed on the tensile stresses in the concrete at working load provided that the prestressing steel is amply protected from corrosion, fire hazard, and failure due to fatigue which might result from extensive cracking and frequent opening and closing of the cracks.

The British Code recommends that the tensile stress can be increased by 250 lb. per square inch, and the compressive prestress at the edge of the member where this stress is greatest should be not less than 1500 lb. per square inch; in the writer's opinion this stress is unduly great.

Factors of Safety.

It is well known that it is necessary to investigate the conditions both at working load and at failure of a prestressed concrete member. This requirement is in conformity with the recommendations of the Code[1] and ensures that known margins of safety are provided at cracking and at failure, although the magnitudes of these margins may be altered when this matter has been considered more fully. The Code recommends the same factor of safety against failure for most forms of construction, namely, 1¼ for the dead load and 2¼ for the imposed or live load, or 2 for the combined load, whichever gives the smaller load. The corresponding factors adopted in the U.S.A. Recommendations[2] are 1·2 for the dead load and 2¼ for the imposed load, or an overall factor of 1·8 for the combined load. In Germany the factor is 1¾ for the combined load. It is preferable for the factor to be varied to take into account such matters as the probability of overloading, the danger to life in the event of failure, and the economic consequences of failure. For example, in a structure containing liquid, it is possible to increase the live load only by storing a liquid heavier than that assumed in the design and a smaller factor of safety with respect to the live load might therefore be employed.

The possibility of inaccuracies in design and construction makes it essential to use a factor of at least 1½ for the combined load.

The deformation of the structure should also be considered. For example the deformation permissible at working load should not be such that the serviceability of the structure is impaired, but the structure should not be so rigid that little deformation occurs as failure is approached. The ratio of the factor of safety against failure to that against cracking should therefore, in the writer's opinion, be at least 1½. If the ratio is less than, say, 1¼, little warning of impending failure would be given; furthermore the structure would offer little resistance to impact and therefore its ability to absorb shock (due to an explosion, for example) would be small.

Three types of structure are considered in an early report,[5] (1) Structures in which cracking is not permissible at working load; (2) Structures in which cracking is permitted at abnormal loads provided tensile stresses do not occur at ordinary working load; (3) Structures of minor importance in which cracking is permitted at ordinary working load. A structure of type (1) may be either (a) a structure in which no tensile stress is permitted at working load, or (b) a structure in which the tensile stress is less than that corresponding to visible cracking, in which case the factor of safety against cracking is less than for a monolithic structure of type 1(a). Structures of types (1b) and (2) are suitable when a considerable degree of elasticity is desired before failure occurs, and the ratio of the factor of safety against failure to that against cracking is large.

The factor of safety against failure and the serviceability at working load of the structure should be considered as equally important. If the factor of safety against cracking is required to be large, that against failure should also be increased to ensure that visible warning of overloading is given.

The conditions described in the foregoing relate to statically-determinate structures. In the case of statically-indeterminate structures there may be further complications, but the permissible stresses are the same although some redistribution of stress may occur. Secondary bending moments, which may occur due to the profiles of cables, may reduce the counteraction due to the prestressing steel under conditions of working load, and positions of cables which are suitable for the working load may not be suitable for conditions at failure. It has been suggested that the conditions at failure are not affected by the position of the prestressing steel and that plastic hinges form in positions that ensure a satisfactory factor of safety against failure. This behaviour has not yet been satisfactorily proved but if it did occur such a solution would be undesirable, since the position of the prestressing steel would be outside the tensile zone and large cracks would develop in the event of overloading. Moreover, early failure due to shearing forces would occur in members subjected to rolling loads, since the resistance to diagonal tensile stresses is reduced when the distance of the steel from the compression edge is reduced. A member designed for conditions at failure only may suffer from the same disadvantages as a statically-determinate structure, namely, excessive deformation and cracking at working load. It is therefore necessary to consider conditions at working load and at failure. The prestressing steel should be in the tensile zone, where it is most effective at failure and for the prevention of wide cracks if overloading occurs. The conditions at working load should be investigated, the permissible stresses, as discussed in the

previous article, being such that excessive deformation and cracking is avoided. Diagonal cracks may occur as a result of a combination of bending and shearing stresses where inclined cables or stirrups are not provided. Either a small bending stress or a small shearing stress may be dangerous if the other stress is large.

Unless good supervision and the fulfilment of the assumptions made in the design can be ensured, the use of reinforced concrete (with or without a small prestress to reduce cracking) is preferable to prestressed concrete. Unless the maximum possible losses are assessed, the calculated stresses will bear little relation to those which may occur and the adoption of conservative stresses is not in itself sufficient to ensure a safe structure.

It is particularly desirable that conditions at transfer should be assessed closely, since the adoption of excessive compressive stresses may cause immediate failure, either local or complete, if the specified strength of the concrete is not obtained; if the difference between the strength required and that obtained is not so great, such stresses may cause losses greatly in excess of those expected, with a consequent reduction in the factors of safety against cracking and against failure. No advantage is gained, however, by providing a greater prestress than is necessary, since this leads to an increase in the rigidity of the structure, and affects adversely its resilience. The bending moment M_{cr} causing cracking is $Z\ (f_{ce} + f_{ct})$, in which f_{ce} is the effective prestress and f_{ct} the stress at which cracks become visible. The stress f_{ct} will usually vary between 900 lb. and 1100 lb. per square inch if pre-tensioned steel is used, and between 600 lb. and 1000 lb. per square inch if post-tensioned steel is used, depending on the distribution of the steel, the strength of the concrete, and the efficiency of the bond. If a prestressed structure is composed of separate precast blocks, no tensile resistance should usually be taken into account, but to obtain the greatest possible value of M_{cr} a modulus of 300 lb. per square inch may be allowed if there are satisfactory mortar joints. The foregoing points must be considered when deciding on suitable permissible stresses. It is also necessary to ensure that the risk of sudden failure due to shearing, principal stress, or to compression is eliminated.

REFERENCES.

1.—British Standard Code No. 115 (1959): "The Structural Use of Prestressed Concrete in Buildings".
2.—"Tentative Recommendations for Prestressed Concrete." Journal of American Concrete Institute, January, 1958.
3.—"Spannbeton,—Richtlinien für Bemessung und Ausführung." (DIN 4227, 1953.)
4.—"P.C.I. Standard Building Code for Prestressed Concrete (Tentative)." Prestressed Concrete Institute, Chicago, (1959).
5.—"First Report on Prestressed Concrete." Institution of Structural Engineers (1951).

Design of Motorways.

THE Ministry of Transport stated recently that when a motorway passed through more than one county, the design and supervision of construction would be entrusted to a firm of consulting engineers, thus ensuring the necessary unified control.

A spokesman on behalf of the County Councils Association has stated that the county councils are convinced that they can build motorways cheaper than anybody else since they have been building trunk roads for seventy years. It is feared that if in the long run the major works in road-building go to consultants, so would the best men. The local authorities would then be faced with having to do their own road-building and maintenance of the motorways with second-rate staffs.

The Terminology of Prestressed Concrete.

THE result of the competition recently held by this journal for a word to describe prestressing steel and other means of precompressing concrete has evoked some interest, as is shown by the correspondence in the following.

MR. K. K. MCKELVEY of Messrs. W. S. Atkins & Partners writes as follows:

"With reference to the competition you have recently held to find a word to describe prestressing materials, may I say that the word 'strictor' which you suggest appears quite suitable. Your original enquiry asked for a word which could be used in parallel with reinforcement. Although it is a noun, the latter has a verb as a derivative. It would seem appropriate, therefore, to devise a similar series of derivations for the new word. I should appreciate having your comments on this. Can one, for instance, talk about 'strictored concrete', or 'the operation of strictoring', and so forth?"

[As is indicated in the first sentence of this letter, the only purpose of the competition was to find a suitable word to describe the various media used to precompress (or prestress) concrete and which at present are mainly steel wires, steel-alloy bars, and wire ropes, and less frequently jacks and the like, but which may include other materials and means in the future. If such a generic word were used, the terminology of prestressed concrete would be similar to that of reinforced concrete, in which the word reinforcement is used to describe all the various types of steel and other materials used for the purpose of enabling a structural member of concrete to resist tensile forces. If the term "prestressed concrete" and precise phrases such as "tensioning the steel (or strictor)" are used, there is no need for a term such as "strictored concrete" or a phrase such as "the operation of strictoring".—ED.]

M. HENRI LEMOINE of the Société Technique pour l'Utilisation de la Précontrainte writes as follows:

"In the Editorial Notes of your number for March 1960 you state 'The term prestressed concrete is itself unsatisfactory because it does not indicate the kind of stress to which the concrete is subjected'. We have been dealing with the promotion of prestressed concrete for fifteen years in most countries and it is the first time that we have seen that term criticised. We would like, therefore, to take up the defence of the term 'béton précontraint' (in English 'prestressed concrete') created by M. Freyssinet, who always insists, with reason, on the generality which must be attached to that term.

"It is wrong to consider that the idea of prestressing is concerned with the introduction of compressive stresses only. It is common, for example, that in the case of a freely-supported beam the results of prestressing are compressive stresses in the lower part and tension in the upper part. This tension is commonly accepted because the imposed loads can only decrease it and change it to compression.

"Therefore, we consider that prestressing is the proper term to designate, in all its generality, the effect of the forces to which a structure is submitted by the engineer before the application of the loads in order to resist better the stresses resulting from these loads."

[Although we have on more than one occasion expressed the view that the term "prestressed" and its derivatives are unsatisfactory, for the reason stated, they are now in such common use that it would be unwise, and probably impossible, to change them. We have always thought that the French term "précontrainte" is much more descriptive of the action of precompressing, or constraining, the concrete and that the term "prestressed" is not a literal translation.—ED.]

In the last line of Editorial Notes under the heading "A Word is Wanted" in our number for March, 1960, the word "rigidity" should be replaced by "resistance to bending". We are indebted to Mr. A. Goldstein, B.Sc., M.I.C.E., for drawing our attention to this matter.

Symposium on Concrete in Hot Countries.

THE Reunion Internationale des Laboratoires d'Essais et de Recherches sur les Materiaux et les Constructions (RILEM) are holding a symposium on "Concrete and Reinforced Concrete in Hot Countries" in Haifa, from July 17. Details are obtainable from the Building Research Station, POB 4910, Haifa, Israel.

FIG. 1.

New Large Bridges in Australia.

A PRESTRESSED BRIDGE OF 320-FT. SPAN.

THE central span of the prestressed concrete bridge (*Fig.* 1) across the Narrows on the Swan River near Perth, Western Australia, is said to be the longest of its kind in the world. The bridge is 1100 ft. long and comprises one span of 320 ft., two spans of 230 ft., and two spans of 160 ft. (*Fig.* 2). The carriageway is 70 ft. wide and on each side is a footpath 10 ft. wide with a protective railing set back 2 ft. from the face of the kerb. The steepest gradient is about 1 in 28. The clearances are 26 ft. for river traffic and 15 ft. over the roads under the end spans. Within the limits set by the requirements of clearances and gradients, the length and proportions of the girders were selected principally for æsthetic reasons.

The bridge is designed to carry 80 per cent. of highway-bridge loading class HA in British Standard No. 153 and vehicles weighing 75 tons. It also carries five 30-in. pipes for water and sewage, gas mains, and electricity and telephone cables. The total imposed load therefore exceeds the full class HA load plus the load from normal services. The bridge is designed to be fully continuous under live load. It is anchored at the north abutment, and all expansion is provided for at the south abutment. The longitudinal girders supporting the deck are carried on reinforced concrete rocking columns at each of the four piers.

Foundations.

The foundations of the piers and abutments comprise 180 piles, known as Gambia piles, which were devised by the consulting engineers. They comprise a steel tube having an outside diameter of 2 ft. 7½ in. and a ½-in. wall, and a conical steel shoe. Before commencing driving reinforced concrete is placed in the lower part of the tube for a depth of 15 ft. Driving is by means of a 10-tons drop-hammer operating inside the tube and striking a removable dolly on the top of the concrete. The piles are extended as required by welding on additional tubes. After completion of the driving the tubes are filled with reinforced concrete. The piles were driven through sands, silts, gravels, and clays and are founded on very firm sand overlying limestone. The lengths of the piles are from 80 ft. to 120 ft.

Only the reinforced concrete core is considered as carrying the load, so that corrosion of the tube does not endanger the structure. Each pile can carry a maximum dead load of 200 tons, which is increased to 250 tons by transient loads. Test loads of 400 tons were supported without measurable permanent deflection and it is thought that each pile is capable of carrying more than 600 tons.

A different type of pile was used at the north-shore pier to allow for possible movement of the reclaimed ground through which the piles were driven. The upper part of each pile for a length of 60 ft. is a precast reinforced concrete column placed inside a steel cylinder of 4 ft. diameter after the cylinder had been

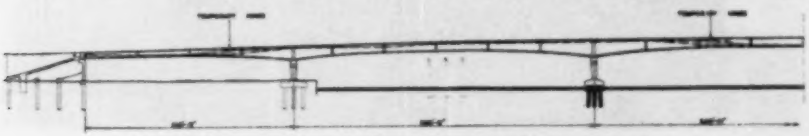


Fig. 2.—Prestressed Bridge: Half Longitudinal Section.

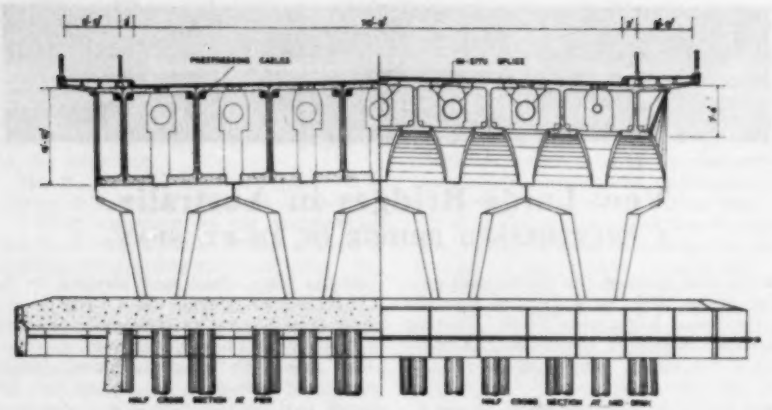


Fig. 3.—Prestressed Bridge: Transverse Section.

driven. The space between the cylinder and the column allows the cylinder, which is weak in bending, to deflect if the ground moves, without moving the column which supports the pier.

Abutments and Piers.

The abutments are of cellular form with a sloping top slab spanning between the interior walls. The purpose of the slab is to prevent an abrupt change in the level of the road due to uneven settlement of the earth supporting the carriageway; the depth of the earth decreases gradually from the full depth at the embankments forming the approach to the bridge to practically nothing at the edge of the deck.

The bases of the two piers on the river banks are simple reinforced concrete pile-caps. For the bases of the river piers (*Fig.* 3) precast concrete slabs served as permanent shuttering and enabled the concrete to be placed in the dry. The slabs conceal the tops of the piles at low water.

Four reinforced concrete columns, each of which supports two longitudinal girders of the deck, are provided at each pier (*Figs.* 3 and 4). Each column is set on

Fig. 4.—Prestressed Bridge: Underside of Deck.

curved bearing-plates of stainless steel and is surmounted by similar plates arranged so that the curves at the top and bottom of each column form part of a cylinder. The columns thereby become in effect parts of large roller-bearings of diameter equal to the height of the column, and allow the longitudinal expansion and contraction of the deck to take place with the minimum of frictional resistance. Transverse expansion and contraction are allowed for by groups of roller-bearings of 7 in. diameter set in boxes in the concrete bases of the piers. The boxes are filled with dense tar to prevent corrosion and the entry of rubbish and water. The bearings are provided under three columns in each row of four.

Superstructure.

The girders of the deck are made up of precast blocks which were joined and prestressed in place. The main girders are designed as double cantilevers with suspended beams and carried their own weight during erection, but act as continuous beams under subsequent imposed loads. Unusual features of the deck are prestressing by means of stranded-wire ropes situated outside the members (*Figs.* 4 and 5), vertical prestressing to increase the resistance to shearing, the method of casting the precast blocks, and the method of obtaining continuity.

The webs and flanges of the I-shape longitudinal girders are as thin as practicable to produce an efficient cross section, and therefore there is insufficient space for the groups of wire ropes required to impose the large prestressing forces required. Consequently the wire ropes imposing the longitudinal prestressing force are placed outside the webs. With this arrangement the thickness of webs and flanges need

Fig. 5.—Prestressed Bridge: Prestressing Steel on Side of Girder.

be only 8 in., since it is necessary only to accommodate the vertical prestressing wire ropes and mild steel reinforcement.

To achieve the greatest eccentricity, and therefore economy, the longitudinal groups of wire ropes should be as small as possible. The overall size of an external group of ropes is governed by the space required between each rope for the placing efficiently of the protective concrete and by the number of ropes. To produce equal force, a small number of large ropes gives a more compact group than a larger number of wires. Tests made on stranded-wire ropes led to the development for this structure of a rope

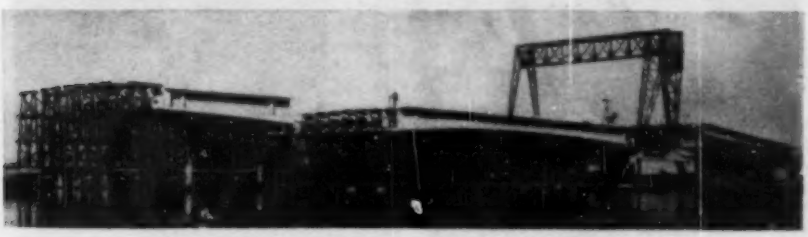


Fig. 6.—Prestressed Bridge: View during Construction.

of 0·7 in. diameter comprising nineteen wires which is used with a split-wedge anchor. At positions where the ropes change direction they pass over lubricated machined-steel saddles which have such a low frictional resistance that the loss of prestress from this cause was only 4 per cent. The prestressing forces were measured accurately with load-cells, the readings being taken on mirror galvanometers. After tensioning, the ropes were surrounded with fine concrete to protect them from corrosion, and to bond them to the webs; mild steel bars were passed through holes in the webs (*Fig.* 5). The arrangement of the wire ropes is shown in *Fig.* 2.

The vertical prestressing ropes are provided to reduce the principal tensile stresses due to shearing on the thin webs, but the ropes alone are not sufficient to resist the total tensile forces due to shearing at the breaking load. Additional mild steel reinforcement was therefore provided. Because of the lack of information on the behaviour of prestressed concrete beams in combined bending and shearing, a model beam of one-third full size was made and tested to verify the assumptions. The model failed at the predicted load, which had been calculated on the assumption that the shearing forces would act as in a lattice beam, and that the main group of ropes was fully bonded. The load-factors were 1·95 on dead load plus imposed load, or 1·5 on dead load plus 4·4 on imposed load.

The blocks forming the girders were made by first casting the bottom flange and the lower part of the web, followed a few days later by the remainder of the block which was cast in a mould bolted to the lower part of the web. The upper mould was of constant size, the variable depth of the web and the width of the bottom flange being accommodated in the lower mould. The blocks were cast in a yard adjacent to the bridge and were transported on bogies travelling on a narrow-gauge track from which they

Fig. 7.—Prestressed Bridge: Beams for Suspended Span ready for Lowering.

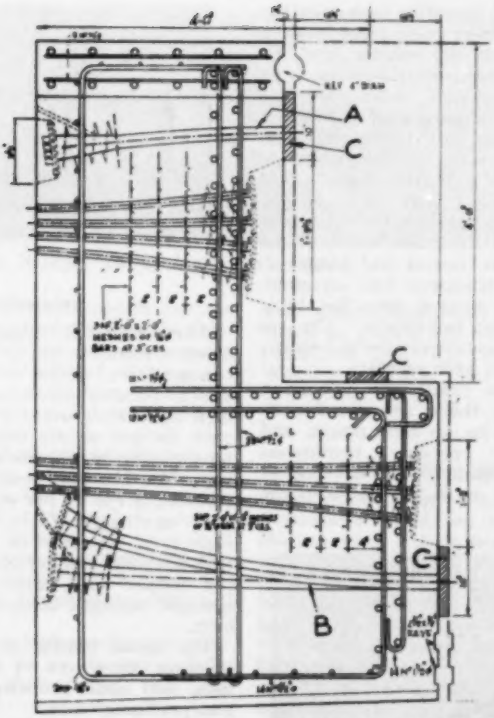


Fig. 8.—End-block at Temporary Hinge of Prestressed Bridge.

were lifted by a travelling goliath crane and placed on a staging formed of jarrah poles driven as piles (*Fig.* 6).

After a complete section of the staging for the double-cantilever girders had been loaded with blocks, the blocks were adjusted for line and level by means of small screw-jacks. The 3-in. spaces between the blocks were filled with fine concrete compacted with immersion vibrators. The primary wire ropes of the groups for the double-cantilever girders are threaded through openings in the transverse webs of the girders and tensioned; the anchors for these ropes are in the lower part of the ends of the girders at the temporary hinges and are therefore inaccessible after the suspended beams are in position.

The suspended beams between the ends of the cantilevers were constructed above their final position (*Fig.* 7) so that there was access to the ends of the beams for tensioning. When prestressing was complete, the beams were lowered on jacks until the halved ends rested on strips of neoprene on the supports at the ends of the cantilevers. The temporary props beneath the girders were then removed, and the secondary ropes of the main groups, which are anchored in the upper part of the ends of the cantilevers, were tensioned.

The transverse ropes were then tensioned, strips of deck slab were cast in place, precast facing slabs were erected, and cantilevered slabs forming the footpaths were concreted. The final operation to complete the superstructure was to make the longitudinal girders continuous by converting the temporary hinges into rigid connections, which was done by caulking with mortar the vertical joints between the precast end-blocks at

the supports and clamping them together by tensioning short wire ropes passing through ducts in the blocks (*Fig.* 8). Each rope comprises twelve 0·276-in. wires and overlaps the main ropes, and thus produces complete continuity. Details of a temporary hinge are shown in *Fig.* 8, in which the ducts for the short ropes are indicated by (A) and (B). Pads (C) of 1 : 1 cement mortar were placed in the gap between the faces of the end-blocks extending across the entire width of the block. When the mortar had hardened, the ropes were tensioned and anchored, and grout was injected into the ducts from behind the end-blocks. The remainder of the gap between the end-blocks was then packed with mortar.

The expansion joints are a German type comprising thick articulated cast-steel plates moving on steel ramps, and are designed to provide a continuous surface for movement while allowing for a variation of 7 in. in the length of the deck.

Fig. 9.—Part Elevation of Prestressed Bridge.

Fig. 1

Stresses.

The calculated stresses in the prestressed members are as follows. In the concrete the initial compressive stress due to bending was 2500 lb. per square inch and under working conditions it is 2300 lb. per square inch. There is no tensile stress in the concrete due to bending. The principal tensile stress due to shearing is 150 lb. per square inch. The crushing strength of 6-in. cubes at twenty-eight days was 8500 lb. per square inch with a standard deviation of about 500 lb. per square inch, compared with the specified strength of 6500 lb. per square inch.

The initial tensile stress in the prestressing ropes was 83 tons per square inch, and under working conditions 63 tons per square inch.

Finishes.

The exposed faces of the outer girders are covered with precast slabs (*Fig.* 9) made with crushed white quartzite, white sand, and white cement. The aggregate is exposed and has a light and rough reflective surface which is said to be particularly effective in the bright sunshine and clear atmosphere of Western Australia.

Fig. 11.—

Fig. 10.—Perspective of Gladesville Bridge: An Arch of 1000-ft. Span.

The facing of the abutments and the cantilevered structures forming the footpaths are constructed in white-cement concrete. The balustrades forming the parapets are of anodized aluminium alloy on a granite kerb. The carriageways, which are finished with asphalt, are flanked by polished granite pilasters at each end of the bridge. Railings are provided between the footpaths and the carriageways.

The bridge contains 10,850 cu. yd. of concrete, 1310 tons of mild steel reinforcement, 1215 tons of steel tube in the piles, and 325 tons of prestressing wires and ropes.

The weights of high-tensile and mild steel in the deck are 7·5 lb. and 19·5 lb. per square foot respectively. The total cost is about £11 15s. per square foot of the gross area of the deck and abutments.

The bridge was constructed for the Commissioner of Main Roads of the Government of Western Australia. The consulting engineers were Messrs. G. Maunsell & Partners, with whom Mr. E. W. H. Gifford collaborated in the design of the prestressed concrete deck. Mr. C. S. Chettoe also advised on the design. The consulting architects were William Holford & Partners. The contractors were Messrs. Christiani & Nielsen of Copenhagen in association with Messrs. J. O. Clough & Son, Pty., Ltd., of Perth, Western Australia. The prestressing was by the Udall system.

AN ARCH OF 1000-FT. SPAN.

The new bridge (*Fig.* 10) over the Parramatta river, an arm of Sydney Harbour, will have a reinforced concrete arch spanning 1000 ft. and, it is believed, it will be the longest single-span concrete bridge in the world. The structure will have a prestressed concrete deck continuous over nineteen spans each 100 ft. long and carried on slender prestressed concrete columns. Over the river, the columns are to be supported on the arch which will comprise four interconnected ribs of box section. Each rib is to be 20 ft. wide, 12 ft. deep at the crown, and 24 ft. deep at the springings. The clear height of the crown above mean water level is to be 142 ft. The abutments are to be of plain concrete and embedded in the sandstone forming the banks of the river. The deck is to have a carriageway 72 ft. wide and two footpaths each 6 ft. wide and will comprise eight prestressed precast beams in each span,

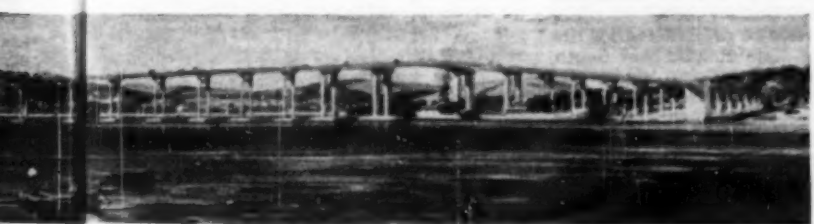


Fig. 11.—…t at Hobart, Tasmania. (*See page 202.*)

supported on reinforced concrete transverse beams across the tops of the columns, and a cast-insitu slab strongly reinforced over the junctions between spans to give continuity under live load. The deck will have sixteen spans of 100 ft. each and a central part 300 ft. long supported directly by the arch. Expansion joints will be provided midway between the crown and the abutments.

The ribs of the arch will be precast in parts weighing about 50 tons each, taken to the site by water, set upon centering, and connected with cast-insitu concrete. To compensate for the combined effects of shrinking, creep, and elastic compression of the concrete, fifty-two groups each of three flat-jacks acting in series will be embedded at each plane of contraflexure in each rib. The jacks will be inflated initially with fluid at a pressure of 2000 lb. per square inch, producing an expansion of about 1 in. in each jack. The fluid will then be replaced by cement grout in one jack at a time. The total jacking movement in each rib will be about 6 in., and since at the time of jacking part only of the shrinking and a negligible amount of creep will have taken place, the arch will be slightly above its final position, thereby facilitating removal of the centering. The working stress in the concrete arch will not exceed 2250 lb. per square inch. The strength of works-cubes of the concrete at twenty-eight days is required to be at least 6000 lb. per square inch.

The prestressed columns will be 10 ft. by 2 ft. in cross section, arranged in pairs, and designed to accommodate expansion and contraction by bending to obviate the need for roller bearings at the column-heads. The columns on the arch will be supported on the transverse beams connecting the ribs, and the columns on the banks will be supported on reinforced concrete foundations on rock.

The bridge is to be constructed for the Department of Main Roads, New South Wales, and is designed by Messrs. G. Maunsell & Partners for the contractors, Messrs. Reed & Mallik, Ltd., (of Great Britain) and Messrs. Stuart Bros., Pty., Ltd., (of Sydney), who submitted a joint tender to build a bridge of their own design. Construction is to begin shortly, and the bridge is expected to be completed in 1962.

A PRESTRESSED CONCRETE VIADUCT 4600 FT. LONG.

The prestressed concrete road bridge (*Figs.* 11 and 12) to be constructed for the Tasmanian Government over the River Derwent at Hobart is to be a multiple-span structure 4649 ft. long, which includes approach spans having a total length of 425 ft. on the western bank and 840 ft. on the eastern bank. The middle part of the bridge between the main abutments is to be 3364 ft. long and will comprise a main navigation span 310 ft. long, two adjacent navigation spans each of 197-ft. span, thirteen spans

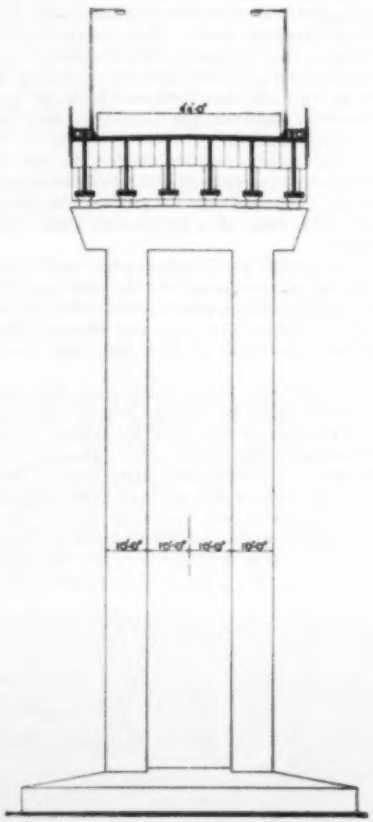


Fig. 12.—Cross section of Viaduct at Hobart.

each 140 ft. long on one side, and six similar spans on the other side. The main span will provide a clearance of 150 ft. above the water for a width of 180 ft., and will comprise suspended beams supported on cantilevers extending from the adjacent navigation spans. Expansion joints are to be provided at one end of the suspended span and at the ends of the navigation spans. The spans are to be continuous over all other supports up to the main abutments.

The carriageway is to be 44 ft. wide and there will be two footpaths each 4 ft. 6 in. wide. Each span will comprise six prestressed precast beams placed longitudinally and a reinforced concrete slab cast in place. The beams, which are to be of I-section, the top flanges forming part of the deck (*Fig.* 12), are to be carried on reinforced concrete transverse beams cast in place at the supports, which are to be slender concrete columns supported on piers cast in place. The piers are to be provided with precast skirtings at the level of the water and each is to be founded on a group of vertical and raking bored reinforced concrete piles of 4 ft. 6 in. diameter. The piles are to pass through the alluvial deposits to the underlying rock. The greatest depth of water is about 120 ft. The piles will be up to 170 ft. long.

The four hollow rectangular piers supporting the navigation spans are designed to resist horizontal thrusts in all directions. The comparatively large movements of the deck resulting from changes of temperature, shrinking and creep will be absorbed by flexure of the columns, which will be precast and prestressed with post-tensioned steel-alloy bars. The columns will be fixed at the piers and have concrete hinges at the top.

The main western abutment is to be in shallow water and is to be a massive reinforced concrete structure founded on the rock. To the west of this abutment the approaches are to comprise a number of short spans of prestressed precast concrete supported on reinforced concrete piers founded on rock, and will form a number of elevated road-crossings. The eastern abutment is to be similar to the western but is to be on the river bank where the rock is near the surface. The eastern approach is to comprise twelve 70-ft. spans of reinforced and prestressed concrete.

The superstructure over the river is to be assembled on a temporary steel staging moved on pontoons from one position to the next. The piles are to be guided and positioned by floating steel templates anchored to the bed of the river and also forming the shuttering for the pile-caps.

The erection of the superstructure will be carried out using steel trestles on pontoons, which will be floated out to the site of the bridge where the trestle will be supported on the pile-caps. The lifting plant will be capable of dealing with precast beams and columns up to 100 tons in weight.

The consulting engineers are Messrs. G. Maunsell & Partners, with whom Messrs. E. W. H. Gifford & Partners are associated for the design of the deck. The main contractors are Messrs. Reed & Mallik, Ltd., and Messrs. Braithwaite & Co., Engineers, Ltd.

Works of Engineering Construction.

REGULATIONS extending the definition of the term " work of engineering construction " in the Factories Act, 1937, come into operation on 15 May, 1960. Under these regulations works now to be included are the construction, structural alteration or repair (including re-pointing or repainting), or the demolition of any steel or reinforced concrete structure (other than a building), roads, airfields, sea-defence works, river works, and civil engineering or constructional works of a similar nature. The definition will not apply to these works if they are carried on in a factory or on a railway or tramway. The effect of the regulations will be that the requirements of the Factories Acts about sanitary conveniences, and notifying sites of operations, accidents and industrial diseases will also apply to the works described. The new safety regulations for constructional work, which the Ministry of Labour proposes to make, will also apply.

FIG. 1.

A Prestressed Railway Bridge in India.

THE new bridge over the River Singhrana (*Fig.* 1) comprises eight spans, each 40 ft. long between the faces of the piers, and carries four broad-gauge main-line tracks which provide access to the new steel works at Durgapur.

The piers are of brick and have reinforced concrete caps supporting the superstructure, which comprises for each of the four tracks (*Fig.* 2) two prestressed precast I-beams (B) carrying a reinforced concrete deck slab (C). The piers are wide enough to support the superstructure of an additional track. The top flange of each beam is 24 in. wide, 6 in. thick at the edge and 10 in. thick at the web, which is 8 in. thick. The bottom flange is 24 in. wide, 12 in. thick at the edge and 19½ in. thick at the web. The deck slab is keyed to the beams by steel bars and castellations above the top flange, and acts with the beams as a composite member. Transverse stiffeners (A), which were cast in place, are placed at intervals between the beams.

Twelve steel-alloy prestressing bars of 1¼ in. diameter are provided in each beam. Nine bars would have sufficed for the working load, but the load causing failure was required to be not less than twice the dead load plus the live load, or the dead load plus three times the live load ; the latter condition prevailed, and the bars were therefore stressed to about 33 tons per square inch instead of 42 tons per square inch, which is normally permissible in these bars.

The beams were precast on timber centering adjacent to the piers. Centering for four beams and shuttering for the sides of two beams were provided. The shuttering was removed from the sides of the beams twelve hours after casting and the beams were allowed to mature for six days before stressing four bars in each beam, which were sufficient to resist the stresses due to the dead load. After this initial prestressing, each beam was moved laterally to its final position and the centering prepared for the next beam. The remaining eight bars were stressed after casting the deck slab; four bars are

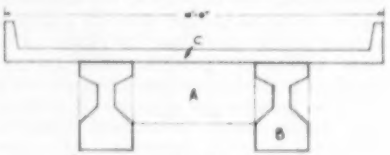


Fig. 2.

anchored at the ends of the beam and four at intervals along the top of the deck.

The sixty-four beams in the entire structure were cast in twenty-eight weeks. The average time from the erection of shuttering to the final prestressing was twenty-nine days for each span. The cost of this method of construction was about £1450 per span for each track. The cost of the construction with mild-steel plate girders was estimated to be £1725 per span for each track.

The bridge was designed by Mr. B. K. Panthaky and the contractors were Shib Banerjee Properties and Construction (Private), Ltd., of Calcutta.

Building Centre at Manchester.

A BUILDING Centre was opened in May at 113 Portland Street, Manchester. The Centre, which is affiliated to The Building Centre, London, will provide a permanent exhibition of building products and materials and a source of information for the building profession and industry.

Slabs Spanning in Two Directions Designed by the Yield-line Method.

SYMMETRICAL CONCENTRATED LOADS.

By G. M. MILLS, B.Sc.Tech., A.M.I.C.E., A.M.I.Struct.E.

In determining the resistance of a slab to failure the principle of superposition does not apply to the combination of two or more systems of loading. The following method of design is based on the relationship between a concentrated load and a distributed load. If a concentrated load is imposed symmetrically on a slab, the uncorrected coefficients for the resulting bending moments can be obtained from the non-dimensional charts in *Figs.* 1 and 2. For small deviations from symmetry, the error in using the charts is negligible. For irregular loads, a method of trial and error, using equations of work, would produce a more accurate solution.

Load Concentrated on a Small Area at the Centre of a Panel.

In an article in this journal for March 1960, the writer describes methods of obtaining values of the factors i and μ for a panel of slab carrying a uniformly-distributed load. These factors are also suitable for slabs subjected to loads concentrated on a small area at the centre of a panel. From these factors, the reduced lengths of l_{xr} and l_{yr} of the sides can be calculated and hence the term $\frac{l_{yr}}{l_{xr}\sqrt{\mu}}$ can be evaluated.

If W_P is the magnitude of the concentrated load, the non-dimensional parameter $s = \frac{W_P}{w.l_x^2\sqrt{\mu}}$ and *Fig.* 1 shows the uncorrected value for β_x. The value of the correction which must be made to allow for corner effects is obtained by multiplying the correcting factor for a uniformly-distributed load by

$$\left[1 + \frac{1{\cdot}35}{1 + s}\right].$$

EXAMPLE.—A panel of a slab is continuous on all sides and is supported by beams 12 in. wide spanning between columns at 25-ft. intervals in both directions. It is required to support a load of 120 lb. per square foot in addition to its own weight and a concentrated load of 6 tons suspended anywhere from the underside of the slab. Calculate the bending moments at midspan and at the supports.

If the slab is $7\frac{1}{2}$ in. thick the total load is (120 + 94) lb. per square foot. Assume $i = 1{\cdot}25$ and $\mu = 1{\cdot}0$. Since the effective span is 24 ft.,

$$l_{xr} = \frac{2 \times 24}{2\sqrt{1 + 1{\cdot}25}} = 16 \text{ ft.}$$

and $\frac{l_{yr}}{l_{xr}\sqrt{\mu}} = 1$. Then $s = \frac{6 \times 2240}{24 \times 24 \times 214} = 0{\cdot}11$.

Therefore, from *Fig.* 1, $\beta_x = 0{\cdot}058$. If no additional corner reinforcement

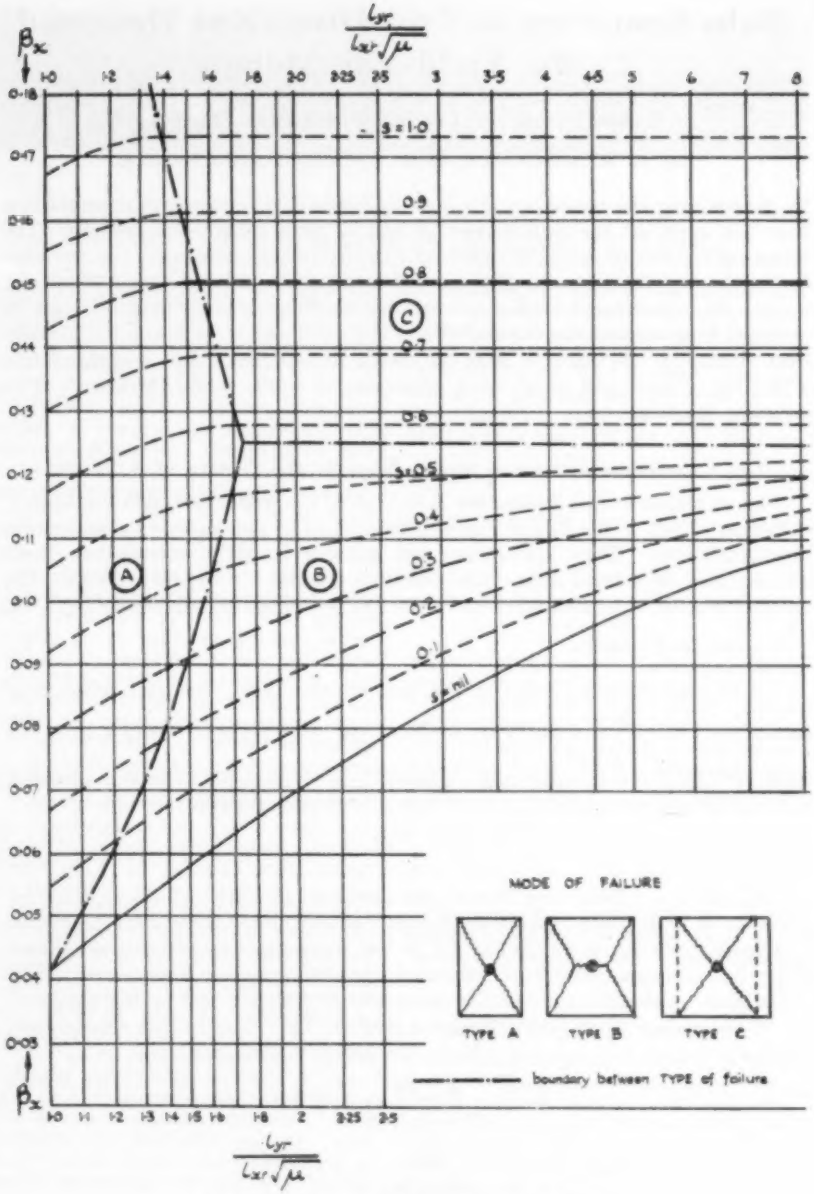


Fig. 1.—Symmetrical Concentrated Load.

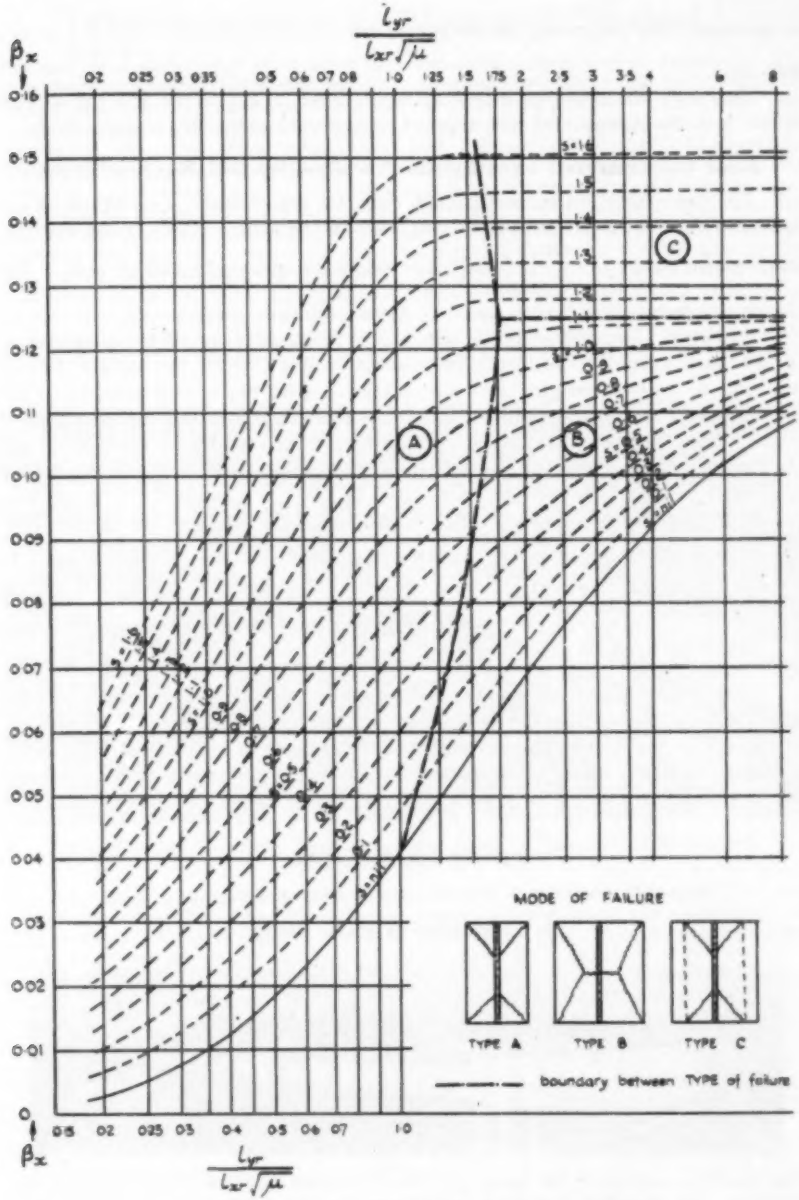


Fig. 2.—Load Concentrated on a Narrow Strip.

is provided the correcting factor is 9·1 per cent. $\times \left[1 + \frac{1{\cdot}3 \times 0{\cdot}11}{1 + 0{\cdot}11}\right] = 10{\cdot}2$ per cent.

Therefore the midspan moment, $M_x = 0{\cdot}058 \times 214 \times 16^2 \times 1{\cdot}102 = 3500$ ft.-lb. and the moment at the support is approximately $iM_x = 4375$ ft.-lb.

Load Concentrated on a Symmetrically-disposed Narrow Strip.

The same notation is used except that, for convenience, l_x is taken as the span in the direction of the longitudinal axis of the load. If W_L is the load per unit length, then $s = \frac{W_L}{w.l_x\sqrt{\mu}}$ and *Fig.* 2 shows the uncorrected value of β_x. The correcting factor is the same as that for a uniformly-loaded slab.

EXAMPLE.—A panel of a slab, which is 15 ft. by 20 ft. and is freely supported on all sides, carries a uniformly-distributed load of 80 lb. per square foot in addition to its own weight, and also carries a 9-in. brick wall parallel to the 15-ft. sides and centrally between them. Calculate the resulting bending moments if the weight of the wall is 1000 lb. per foot. If the slab is 6 in. thick, the total load is 80 + 75 = 155 lb. per square foot, and the extra load due to the wall is $1000 - (\frac{3}{4} \times 80) = 940$ lb. per foot.

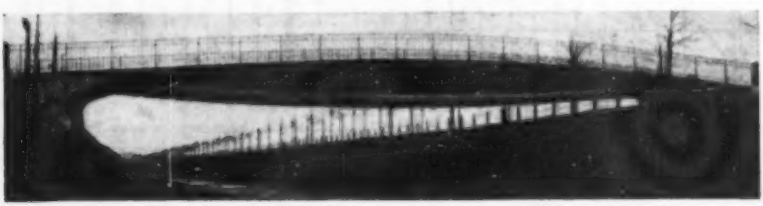
From the nomogram in *Fig.* 2 on page 112 of this journal for March 1960, for $F = 1$ and $\frac{l_{yr}}{l_{xr}} = 1{\cdot}33$, $\mu = 0{\cdot}30$. Therefore, $\frac{l_{xr}}{l_{xr}\sqrt{\mu}} = \frac{1{\cdot}33}{\sqrt{0{\cdot}3}} = 2{\cdot}44$ and $s = \frac{940}{155 \times 15 \times \sqrt{0{\cdot}3}} = 0{\cdot}74$. From *Fig.* 2 on page 207 of the present number, $\beta_x = 0{\cdot}107$.

From *Fig.* 3 (page 115, March 1960) it can be seen that if the corners are held down, corner effects may be prevented by additional reinforcement at the top of the slab at the corners and in each direction capable of resisting half the corresponding bending moment at midspan. If such reinforcement is provided, $M_x = 0{\cdot}107 \times 155 \times 15^2 = 3710$ ft.-lb. and $M_y = 1110$ ft.-lb.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", May, 1910.

A Footbridge of 85-ft. Span.



THE illustration shows a reinforced concrete footbridge over the Weissoritz at Cotta-Dresden. The structure comprises two longitudinal fish-bellied girders each of 85-ft. span.

The Approach to Blackwall Tunnel.

THE new road works (*Fig.* 1) required to improve the approach to Blackwall Tunnel, London, which is being duplicated, include a dual carriageway northward from the tunnels to pass under East India Dock Road to which access will be gained by means of a clover-leaf type of junction.

A bridge of two spans and a length of 55 ft. will carry the east–west road over the approach road and is of structural steel and reinforced concrete. To maintain the flow of traffic westwards and to avoid interference with two 48-in. high-pressure gas mains, the bridge is being constructed in two stages. Upon completion of the first part the gas mains were replaced by four 36-in. welded steel pipes incorporated in the deck of the new bridge. The first stage of the bridge was completed and opened for traffic in August 1959, and the second stage is expected to be complete during the spring of 1960.

The bridge is designed for the abnormal load specified by the Ministry of Transport. The foundation comprises a reinforced concrete slab 5 ft. 6 in. thick extending under both abutments and the central reinforced concrete pier. The excavation for the first part of the bridge was about 30 ft. deep and, as it was about 4 ft. from one of the gas mains, vibration due to driving steel sheet-piles was not permitted. Therefore a trench was excavated in two stages each 15 ft. deep and the piles were lowered at each stage and strutted temporarily against the "dumpling" of earth left in place. The heads of the piles were then tied by means of steel ropes passing under the road to anchorages formed by steel tubes sunk 25 ft. into the ground and filled with concrete. The horizontal ducts for the passage of the ropes were formed by thrust-boring under the road to avoid interference with traffic. The ropes were tensioned by hydraulic jacks before removing the temporary struts and excavating the dumpling.

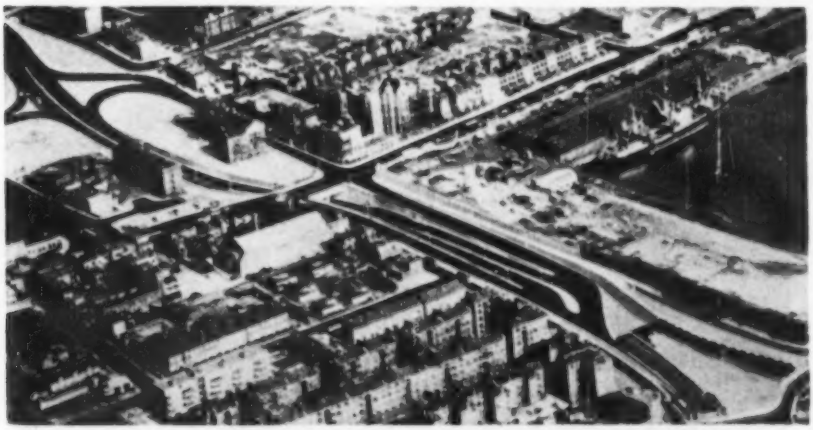


Fig. 1.—The Northern Approach: Entrance to Tunnel in Foreground.

Precast concrete interlocking slabs were used at the sides of the excavation and were left in position to protect the asphalt forming the waterproof membrane.

Retaining walls are necessary on either side of the approach road, which is below the level of the ground where it descends to and ascends from the tunnels. The two walls, and the slab (*Fig.* 2) which is the sub-base of the road, form a large monolithic reinforced concrete channel. All this work is massive in order to prevent flotation in the waterlogged ground. At one part the bottom slab is 6 ft. 6 in. thick and overlies a layer of asphalt which extends up the backs of the walls. Where the sides of the

excavation could be sloped, the asphalt was applied after the wall had been constructed. Elsewhere the asphalt was applied to a permanent lining of the excavation and the concrete of the wall was placed against it. Generally the excavation and construction proceeded between temporary steel sheet-piling (*Fig.* 3). In the deeper sections, the struts supporting the piling were compressed initially to prevent movement when the ground was removed from in front of the sheet-piling. Where the excavation was close to two tenement buildings, 19-in. bored concrete piles placed closely together provided a temporary support to the ground.

The level of the ground-water along the eastern side was 8 ft. higher than was expected, due probably to water leaking from the adjacent dock. The level was not constant over the site, probably because of the non-uniformity of the ground, which is mainly clay. A row of well-points was therefore sunk along the eastern side. A permanent pumping station with a deep pump-well is provided to deal with surface-water collecting in the low-level approach. Water-bars are provided in all joints in the retaining walls.

The retaining walls are faced with Portland stone, and the pilasters at the entrance to the new road are faced with Westmorland slate. The works include 70,000 cu. yd. of excavation and 22,000 cu. yd. of concrete. In general the concrete was mixed on the site in a central batching and mixing plant, but ready-mixed concrete was used in some parts. The plant, seen in the background of *Fig.* 2, comprises two 14-cu. ft. non-tilting drum mixers, a bucket-elevator for loading the aggregates into the main hoppers, and a bucket-conveyor to supply the elevator. The controls of the mixers are operated by compressed air by means of which the loading hopper and discharge chute are raised and lowered. The men required to work the plant comprised an operator at the controls, and two men who deal with the cement (which is generally sulphate-resistant cement supplied in bags) and operate the bucket-conveyor, the chain of which is driven by an

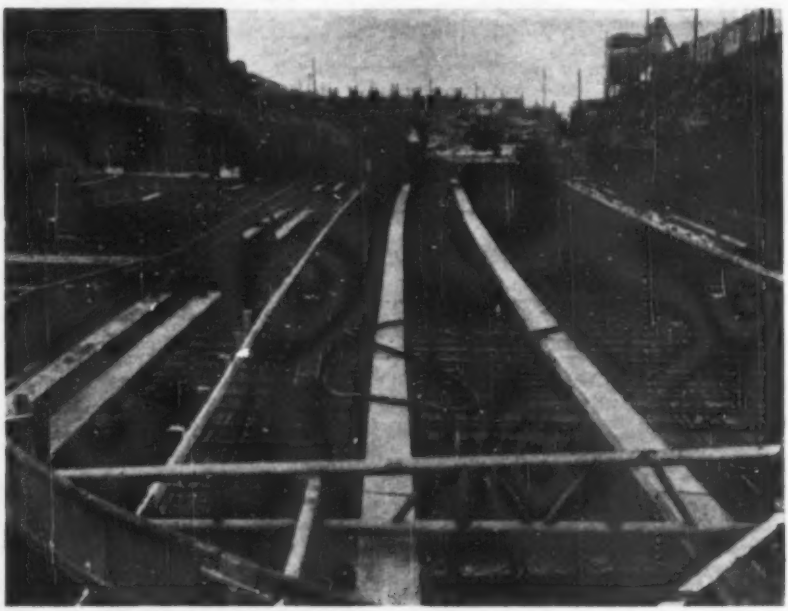


Fig. 2.—Construction of Carriageway.

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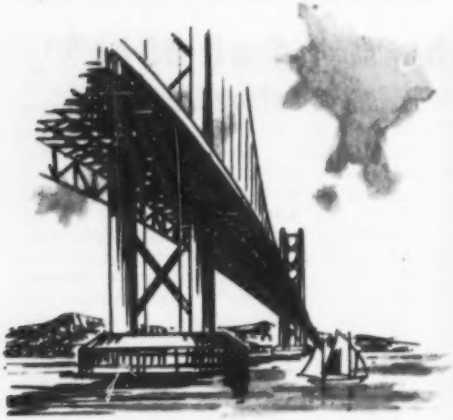

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Fig. 3.—Construction of Retaining Walls.

electric motor ; the slewing movements are operated hydraulically. A smaller concrete plant comprised a 14-cu. ft. non-tilting drum mixer and a portable silo for the storage of cement delivered loose. Concrete was distributed generally by dumpers, but skips on mono-rails were used for the bottom slabs. Vibrating spades and immersion vibrators were used to compact the concrete.

The work, the cost of which will be about £750,000, was designed and carried out under the direction of Mr. J. Rawlinson, C.B.E., Chief Engineer of the London County Council. The contractors are Messrs. Holland & Hannen and Cubitts (Great Britain), Ltd., and Messrs. Fitzpatrick & Son (Contractors), Ltd. The bored piles were constructed by the Cementation Co., Ltd.

The Collapse of the Malpasset Dam.

THE report of the Ministry of Agriculture (of France) relating to the bursting of the reinforced concrete dam near Fréjus in the south of France was issued recently. The dam was completed in 1957 and was claimed to be the " thinnest " dam in the world. It was an arch dam, the radii of the downstream and upstream faces of which varied with the height. At the crest, the upstream radius was about 345 ft., the angle subtended was 135 deg., and the thickness was about 5 ft. The greatest height of the dam above the bed of the river was about 200 ft. At one end, the structure was connected to the rock foundation by a concrete shoulder. The Malpasset dam collapsed on December 2, 1959, after a period of heavy rains and gales. Failure occurred shortly after the reservoir had been filled for the first time.

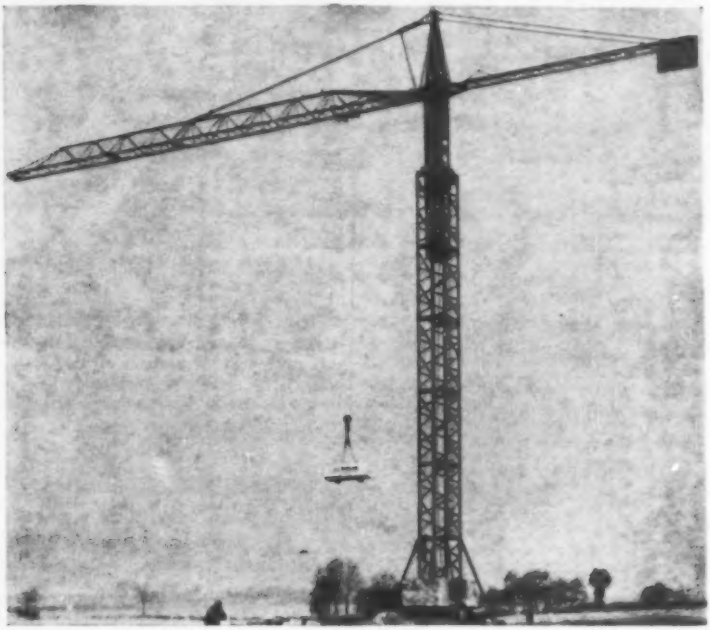
The report exonerates the designers of the dam and the contractors. The calculations on which the design was based were, it says, sound, and both the dam and the shoulder were correctly constructed. The floodgates operated properly. From the fragments of débris left behind by the waters, it is clear that the rock under the dam was faulty. The inquiry is to continue in order to find out exactly what happened to the rock, and to determine how similar disasters can be avoided.

A Sand and Gravel Handbook.

THE Sand and Gravel Association of Great Britain has published a book entitled " Sand and Gravel Handbook ". In addition to a list of the members and a map showing the location of the pits worked by the members, much data relating to the materials are given including abstracts from relevant British Standards. The information is mainly intended for pit managers and the like, but engineers and surveyors of local authorities, consulting engineers, and particularly contractors and makers of precast concrete should find this book a useful work of reference.

The handbook can be obtained from the Sand and Gravel Association of Great Britain at 48 Park Street, London, W.1 (price 20*s*.).

A Large Tower-crane.



THE illustration shows a tower-crane which is claimed to be the largest of its kind in the world, and in the first instance is to be used on the construction of a water-purification works near Eccup reservoir, Yorkshire.

The greatest load which the crane can lift is 15 tons at a radius up to 40 ft. The length of the horizontal jib enables loads to be lifted at a radius of 131 ft., at which extent the greatest load is 4 tons. The height of the crane is 185 ft. up to the underside of the hook. In the illustration the tower is erected to give a lifting height of 130 ft. The crane runs on a broad-gauge rail-track. The tower is mounted on four bogies each having three wheels in line. Each bogy is driven by a 4-h.p. motor. The ballast comprises a weight of 50 tons of concrete blocks. Duplicate controls, which are of the push-button type, are provided; one set is in the cabin at the top of the tower, and the other is in a portable panel.

The crane, which is called the "600 Pingon", was manufactured in France, and was supplied by Messrs. George Cohen Sons & Co., Ltd., to Messrs M. Harrison & Co. (Leeds), Ltd., who are the contractors for the water-purification scheme in Yorkshire. A similar crane is in use on the construction of a dam in France.

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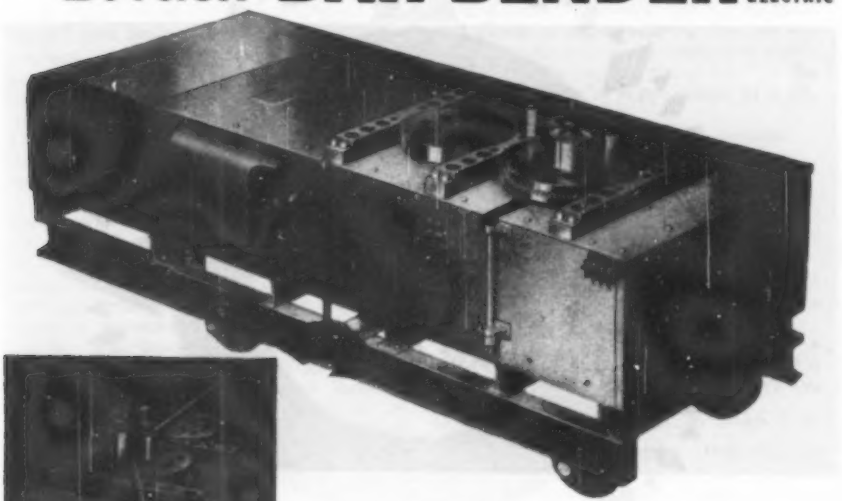

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Code of Practice for Road Bridges in India.

THE second part of a code of practice for the design of road bridges* was published recently by the Indian Roads Congress. The first part, which was issued in 1956, dealt with general features of design; the second part deals with loads and working stresses. Some of the recommendations apply only to conditions in India, but they are of interest, although not of direct application quantitatively, elsewhere. Other data are of value as they are given in greater detail than in the corresponding memoranda issued in Great Britain.

LOADS.—The live load recommended for general use in the design of permanent bridges and culverts comprises a number of eight-wheel lorries each with two four-wheel trailers. The overall length of the lorry and trailers is 67 ft. and the distance between successive lorries and trailers is 60 ft. The transverse distance between the centres of the wheels of lorries and trailers is 6 ft. and the distance between the centres of the nearest wheels of adjacent lorries and trailers varies from 16 in. on an 18-ft. carriageway up to 4 ft. on carriageways 24 ft. or more in width. The loads are 6000 lb. on each of the two leading axles of the lorry, 25,000 lb. on each of the two rear axles of the lorry, and 15,000 lb. on each of the axles of the trailers. The contact areas of the wheels are 10 in. by 20 in. for an axle load of 25,000 lb., 8 in. by 15 in. for 15,000 lb., and 6 in. by 8 in. for 6000 lb.

For temporary bridges the live load is 60 per cent. of the foregoing and the contact areas of the wheels are reduced. A heavier loading represented by a procession of tanks weighing 70 tons each is required for bridges on certain roads. The basic live load on footpaths is 85 lb. per square foot. The live loads on bridges carrying tramways are also specified.

CONCENTRATED LOADS ON SLABS.—An empirical rule for determining the width e of a concrete slab considered to be effective in supporting a concentrated load is given in the form

$$e = kx\left(1 - \frac{x}{l}\right) + W$$

in which x is the distance from the centre of the load to the nearer support; l is the span of the slab (the effective span if freely supported or the clear span if continuous); W is the effective width of the loaded area, that is the width of the contact area plus twice the thickness of road metal or other wearing coat on the concrete slab; and k is a coefficient which depends on the ratio of the width l' of the slab to the span l; some values of k are

$\frac{l'}{l}$	k for freely-supported slab	k for continuous slab
0·5	1·72	1·68
1·0	2·48	2·24
1·5	2·84	2·48
2 or greater	3·00	2·60

Rules are given for cantilevered slabs, for two or more concentrated loads, and for slabs spanning in two directions.

IMPACT.—The effect of impact on a reinforced concrete member of a bridge is allowed for by increasing the ordinary live load by $\frac{15}{20 + L}$ per cent. in which L ft. is the span of the member. Other factors are recommended for exceptional loads.

WIND.—The pressure of the wind recommended to be taken into account in design is 12 lb. per square foot at a height of 10 ft., 20 lb. at 40 ft., 29 lb. at 100 ft., 38 lb. at 200 ft., and up to 48 lb. at 400 ft.; these pressures are greater than the pressure conforming to exposure D of the British Standards. Moreover, in regions where greater wind pressures are prevalent or probable, the recommended pressure is twice the foregoing.

WATER PRESSURE.—The pressure of running water on bridge piers in rivers has to be taken into account and the intensity P lb. per square foot is given by $P = KV^2$, in which V is the speed of the water in feet per second and K is a coefficient depending on the shape of the pier; values of K are 1·5 for a square-ended pier, 0·66 for a circular pier or a pier with semi-circular ends, 0·5 for a pier with a triangular cut-water having an angle not exceeding 30 deg. or 0·7 for an angle of 60 deg., and 0·45 to 0·5 for a cut-water formed of arcs of a circle. In estimating V, the difference of speeds of the water at the bed and surface of the river are taken into account.

OTHER FORCES.—The usual allowances

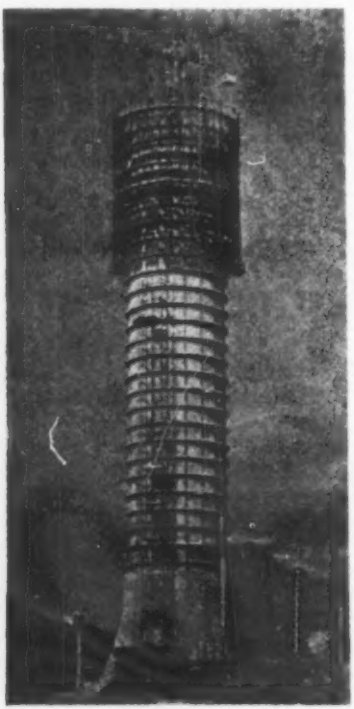
* "Standard Specifications and Code of Practice for Road Bridges. Section II: Loads and Stresses." (New Delhi: Indian Roads Congress. 1958.)

are made for longitudinal forces due to tractive effort, braking, acceleration, and resistance of bearings to movement, and for centrifugal effects in the case of bridges on bends. Recommendations are also given for buoyancy of submerged foundations and for the case of submersible bridges. It is recommended that the range of temperature to be allowed for in the design of reinforced concrete bridges be ±30 deg. F. in regions of moderate climate and ±45 deg. F. for regions where extremes of temperature are experienced. Elaborate rules are given for the calculation of earth pressures on abutments.

For consideration of the effects of earthquakes, the country is divided into regions depending on the probable severity of seismic forces. The coefficients to be applied are zero for bridges in regions where there is thought to be no liability of damage or only minor damage, 0·05 g. where damage is likely to be moderate, and 0·1 g. where severe damage may occur. Greater coefficients are to be assumed in regions where devastating earthquakes have occurred.

A Precast Chimney in France.

A PRECAST concrete chimney 325 ft. high, for discharging radio-active gases from an atomic power station, has recently been constructed at Marcoule, France.



The base has the shape of a three-sided pyramid which gradually becomes cylindrical. The shaft comprises twenty-four rings, each consisting of nine 12-ft. precast panels. The pyramidal base is supported on three hydraulic jacks. A tower crane was erected in the centre of the chimney to lift the panels.

Courses Organised by the British Council.

THE specialist courses organised by the British Council and to be held in 1960, include the following.

Methods of Concrete Usage.—To be held at University College, London, September 18 to 30. The programme will cover, by means of lectures, demonstrations and visits, current methods for prestressed and reinforced concrete; concrete road construction; and an outline of the research being carried on in Britain.

Non-destructive Testing of Materials.—To be held in London, October 16 to 29. The subjects to be dealt with include: research work in progress; the development of industrial methods from the results of research; the development of instrumentation; and the part played by non-destructive testing in modern industrial development.

Particulars of the courses, which are intended primarily for specialists from oversea, are obtainable from the British Council, 65 Davies Street, London, W.1, or from the representative of the Council in other countries.

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Concreting in Hot Weather.

In the construction of a dam at Klang Gates, Kuala Lumpur, Malaya, precautions were taken to cool the concrete before it was placed. The dam is 450 ft. in length at the top and 100 ft. high, and the placing of the concrete occupied seventeen months. The method adopted included the circulation of refrigerated air through the aggregate, the chilling of the mixing water, and the addition of flake ice to the concrete materials.

The refrigerating plant was designed to produce mixed concrete with a temperature of 48 deg. F. when the ambient temperature averaged 85 deg. F. The temperature of the cement, sand, and stone was 85 deg. F., and the temperature of the mixing water was 80 deg. F. The plant was designed for an output of concrete of 22 cu. yd. per hour during a working day of twelve hours. Separate refrigerating plants were used for cooling the aggregates by cold air, for cooling the mixing water, for making ice, and for storing ice. The illustrations show the insulation of the hoppers over the mixers and the delivery pipes.

The air for cooling the aggregate was supplied at a temperature of 35 deg. F. to the compartments of the aggregate bin at the batching plant, separate centrifugal fans delivering the air through cooling-coils in the delivery ducts; other ducts from the top of the tower returned the air to the fan suction-chamber. All the ducts were insulated with cork. The refrigerating plant for this purpose comprised two vertical single-acting enclosed ammonia compressors with a bore and a stroke of 9 in. and driven by 110-h.p. slip-ring motors, a shell-and-tube type refrigerant condenser, a high-pressure receiver, and two centrifugal-condenser water-circulating pumps. The storage bin was divided into six compartments, namely two of 50 cu. yd. capacity for sand and ¾-in. aggregate, three of 33 cu. yd. capacity each for 1¼-in., 3-in., and 6-in. aggregate, and one central circular compartment capable of storing 50 tons of cement. This storage capacity ensured that the coarse aggregate had at least 90 minutes' exposure to refrigerated air. The bin was insulated with two layers of cork 3 in. thick.

A Baudelot chilling system was provided for cooling the mixing water. This

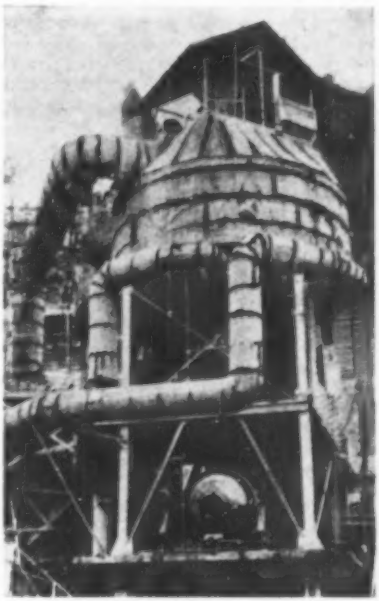


Fig. 2.

Fig. 1.

comprised a galvanised-steel tank over which was a galvanised-steel cooling-coil. Water was pumped to the storage tank and circulated over the cooling-coil by a small vertical-spindle pump. A transfer pump with a capacity of 40 gall. per minute pumped the chilled water to the batching tower. The refrigerating plant included a 5-in. by 5-in. compressor and ancillary equipment.

Ice in ribbon-particle form was supplied by four machines and an ice store was provided below the ice-making plant. For the transfer of the ice to the batching tower a screw-conveyor under the store delivered ice to a bucket-elevator which discharged over a balanced hopper which, in turn, charged a weighing-hopper below. Finally the ice was added to the aggregate and cement in a common collecting hopper. Cork insulation was provided at all stages of the transfer of the ice from the store to the materials bin.

The mixing and batching plant was supplied by Blaw-Knox, Ltd., and the refrigerating plant by York Shipley, Ltd.

The Effect of Temperature on the Creep and Relaxation of Steel Wire.

A REVIEW of the published results of research on the creep of steel wire is given by W. Papsdorf and F. Schweid in a recent number of " Stahl und Eisen ". Since no definite inferences regarding the creep or relaxation of wire over very long periods and at high temperatures can be drawn from these results, tests were made to establish additional data. From these tests it is shown that at ordinary temperatures the rate of increase of strain due to a high constant stress in tempered and heat-treated wire increases initially and then decreases until the strain becomes almost constant. With small stresses this phenomenon was not exhibited in the time available for the tests, and non-tempered wires do not behave in this way.

Tests made on drawn and tempered wire having a tensile strength of about 108 tons per square inch showed that with initial stresses of 55 to 60 per cent. of the tensile strength, losses due to relaxation are greater at temperatures between 22 deg. and 150 deg. C. After long periods or with higher initial stresses the effect of temperature is less. It was also established that the relaxation curves for various grades of steel can be obtained more quickly at high temperatures, and it is stated that it is possible to forecast the effects of creep and relaxation for periods up to a hundred years by means of tests at high temperatures.

It is shown that changes occurred in the physical properties of the tested wires and that these changes can be explained in terms of strain-hardening which are in agreement with theories of creep at normal temperatures.

Revised Code for Tanks and Reservoirs.

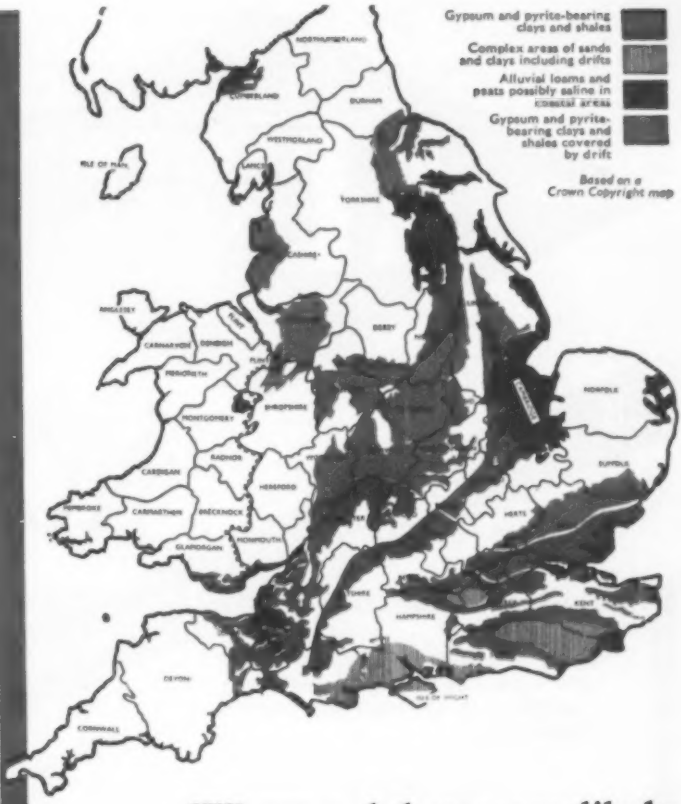
BRITISH Standard Code of Practice CP. 2007 (1960), " Design and Construction of Reinforced and Prestressed Concrete Structures for the Storage of Water and other Aqueous Liquids ",* was published last month and replaces the " Code of Practice for the Design and Construction of Reinforced Concrete Structures for the Storage of Liquids " issued in 1938 (and with revisions or supplements in 1949 and 1950) by the Institution of Civil Engineers. The revised Code applies mainly to tanks and reservoirs, but not to aqueducts or other structures conveying water.

* Obtainable from the British Standards Institution. Price 12*s.* 6*d.*

A New High-tensile Reinforcing Bar.

A NEW high-tensile reinforcing bar is now obtainable in this country. It is a hot-rolled deformed bar having a yield stress of not less than 60,000 lb. per square inch, and complies with British Standard No. 1144 as regards elongation and bend tests. (It also complies with A.S.T.M. Tentative Specification A/305–56T.) The deformations are in the form of circumferential staggered ribs and two longitudinal ribs. The behaviour of the bar is such that the safe bond stress can be assumed to be 25 per cent. higher than that for plain round bars as recommended in B.S. Code No. 114.

The bar is called the " Colbond 60 " and is obtainable in sizes from $\frac{3}{8}$ in. in increments of $\frac{1}{8}$ in. up to $\frac{1}{2}$ in. and of $\frac{1}{4}$ in. up to $1\frac{1}{4}$ in. The net cross-sectional area of an improved form of this bar is the same as that of a plain round bar of the same diameter.


Gypsum and pyrite-bearing clays and shales
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(For example 12,000 lb. per sq. inch at 24 hours and 16,000 lb. per sq. inch at 7 days)

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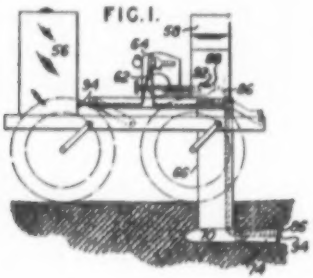
Patent Applications.

Casting Underground Conduits In situ.

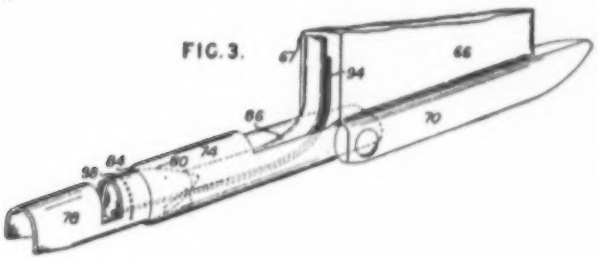
In a method of strengthening a mole drain the drain is partially or completely lined with cement which is applied to the walls of the drain by ejecting it from the expander of a mole drain plough in the form of a powder as the drain is being formed. A spray of water is also ejected from the expander to dampen the cement. The floor of the drain may be left unlined and the strengthening material pressed against the earth walls of the drain by a former. Apparatus for carrying out the method comprises a mole drain plough having apparatus for ejecting cement in the form of a powder from the expander 74. From a hopper 58 to the expander 74 an auger 92 in a feed-box 88 delivers and meters cement blown by an air compressor 62 through a pipe 86, housed in a recess 67 in the rear of the plough blade 66, and through a shaped outlet 84 between the expander 74 and a conical member 80 formed on a former 78 which presses the dampened cement against the earth walls.

FIG. 1.



A pipe 94, in which is connected the pump 64, also in the recess 67, leads from a water tank 56 through the expander 74 into the former 78, and through a slot in the pipe wall water escapes into the cone 80 and then through holes 98 in radial jets, not required, however, in very wet ground. The plough preferably incorporates an automatic depth control system for the bar 70. According to the Provisional Specification the air compressor may be driven from the power-take-off shaft of the tractor drawing the plough or by a separate engine, whereas the flow of cement and/or water may be controlled by valves on the hopper and tank pipes whilst the plough is in the earth, and may be arranged so that the lining is thicker at the bottom than at the top of the drain. —No. 822,315. E. Wood. April 22, 1958.

FIG. 3.



Cement Made Protective against Nuclear Radiation.

A method of treating cement so that when compounded, for example into blocks, protection is afforded against nuclear radiation, comprises adding, during or after grinding, a material to counteract moisture during storage and a proportion of barytes. Preferably 0·01 per cent. to 2 per cent. by weight, of moisture-resistant material and 60 per cent. to 95 per cent. of barytes are incorporated and up to 3 per cent. by weight of gypsum may also be included. A plaster contains 7 parts by weight of barytes and 1 part by weight of a mixture of cement, moisture-resistant additive and gypsum.—No. 826,316. Aspada, Ltd. May 28, 1958.

Plywood Shuttering for Columns.

A NEW form of shuttering for rectangular columns having sides from 6 in. to 30 in. wide comprises plywood fixed to sheet-steel frames (*Fig.* 1). The panels are 3 ft. and 4 ft. long and 16 in., 24 in., and 30 in. wide, and are arranged as shown in *Fig.* 2 for columns of various sizes. The panels are maintained in position by means of tee-head bolts fitting loosely in lugs projecting from the frames, and are fixed rigidly in their correct positions by cotters driven into slots in flanges projecting horizontally. The bolts are then tightened. The boxes are erected one above another and secured by a pair of bolts through each abutting flange.

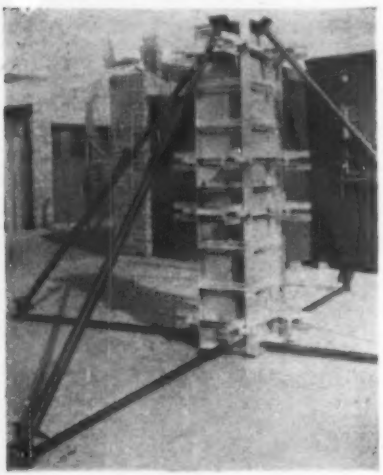


Fig. 1.

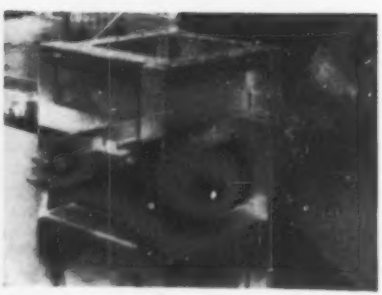


Fig. 2.

In this manner column shutters 6 ft. high and upwards in increments of 1 ft. can be constructed by combining panels of the two standard lengths. The completed shutter is maintained vertically by inclined adjustable steel shutter-props bearing against the top flanges of the panels at one end and, at the other end, against a cleat attached to a scaffold-tube which lies on the ground or floor. The tube acts as a tie since it is secured to the bottom of the shutter by a peg welded to the flange of the panel. The system has been developed by Kwikform, Ltd.

Nomograms for the Design of Beams and Slabs.

IN the article entitled "Nomograms for the Design of Beams and Slabs by the Load-factor Method.—III", which appeared in this journal for April, 1960, the fourth paragraph on page 169 should read as follows.

"When the value of $\frac{M}{bd_1^2 p_{cb}}$ required is greater than the greatest value obtainable from the section (when $\frac{d_n}{d_1}$ is equal to 0·5) reinforcement in compression must be provided; Nomogram No. 4A, 4B or 5 (see Part II) applies when d_s is not less than $\frac{1}{2}d_1$."

The sub-title of Nomogram No. 7 on page 173 should be "Percentage of Reinforcement in Tension".

Skips for Placing Concrete.

IN order to use to the full the lifting capacity of tower cranes, many of the skips used for concreting tend to be lightly made. However, unless metal skips are of robust construction they may have only a short life. In an attempt to meet these conflicting requirements a rubber concreting skip has been developed at the Building Research Station which comprises a tubular steel frame to which a rubber envelope is attached at the top and bottom. The rubber is $\frac{1}{4}$ in. thick and the skip weighs little more than half its counterpart in steel. Trials show that concrete does not adhere to rubber to the same extent as to metal.

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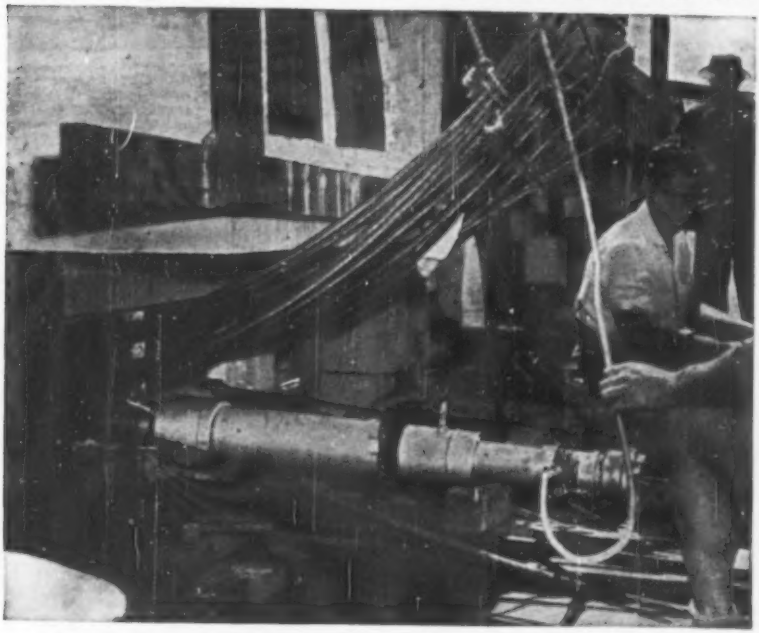
For prestressed concrete



This photograph shows the stressing of multi-wire "Bridge" Strand at the Perth Narrows Bridge in Australia. "Bridge" Strand is made from Somerset Wire.

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